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NATIONAL DAM SAFETY PROGRAM. HINCKLEY DAM (NY 00181), MOHAWK RI--ETC(U)
SEP 78 J J WILLIAMS DACW51-78-C-0035

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1 OF 1

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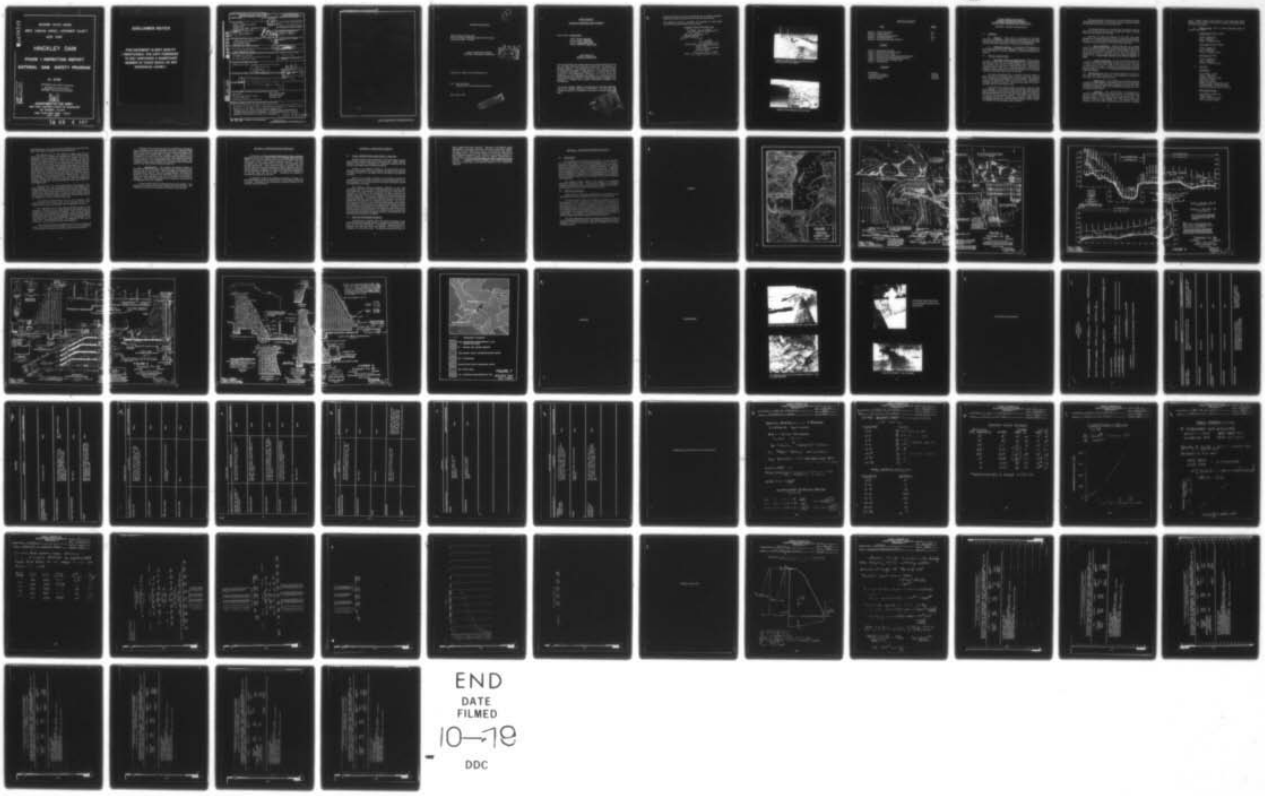

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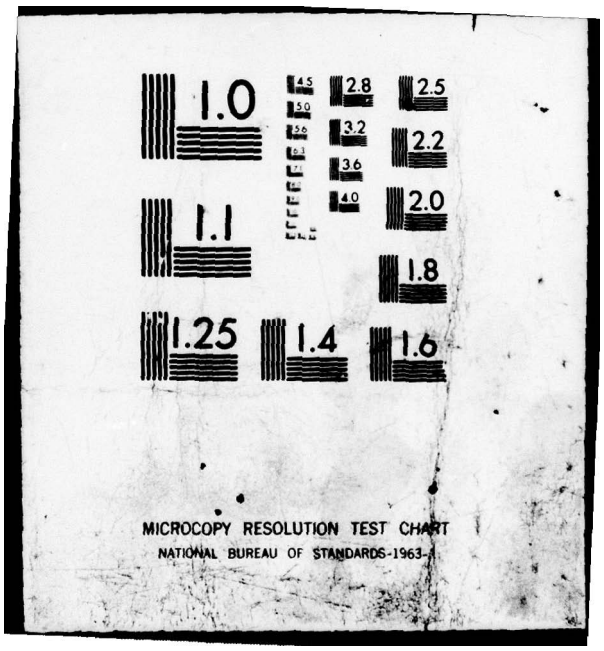
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MOHAWK RIVER BASIN

WEST CANADA CREEK, HERKIMER COUNTY

NEW YORK

HINCKLEY DAM

PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

NY 00181

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CONTRACT NO. DACW 51-78-C-0035

DEPARTMENT OF THE ARMY
NEW YORK DISTRICT, CORPS OF ENGINEERS
26 FEDERAL PLAZA
NEW YORK, NEW YORK 10007

JULY 1978

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	6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s)	8. CONTRACT OR GRANT NUMBER(s)
John J. Williams, P.E.	DACW-51-78-C-0035
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O'Brien and Gere Engineers Inc. 1301 Buckley Road Syracuse, NY 13221	
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National Dam Safety Program. Hinckley Dam (NY 00181), Mohawk River Basin, West Canada Creek, Herkimer County, New York. Phase I Inspection Report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Dam Safety
National Dam Safety Program
Visual Inspection
Hydrology, Structural Stability
West Canada Creek
Herkimer County
Hinckley Dam

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report provides information and analysis on the physical condition of the dam as of the report date. Information and analysis are based on visual inspection of the dam by the performing organization.

Hinckley Dam was found to have areas of seepage with discoloration of standing water at several locations. It was recommended that a boring program be instituted to evaluate the embankment.

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

(1)

MOHAWK RIVER BASIN

Name of Dam: Hinckley Dam
County and State: Herkimer County, New York State
Inventory Number: NY 00181

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Prepared by: O'Brien and Gere Engineers, Inc.

For: New York State
Department of Environmental Conservation

Date: July 6, 1978

PHASE I REPORT

NATIONAL DAM INSPECTION PROGRAM

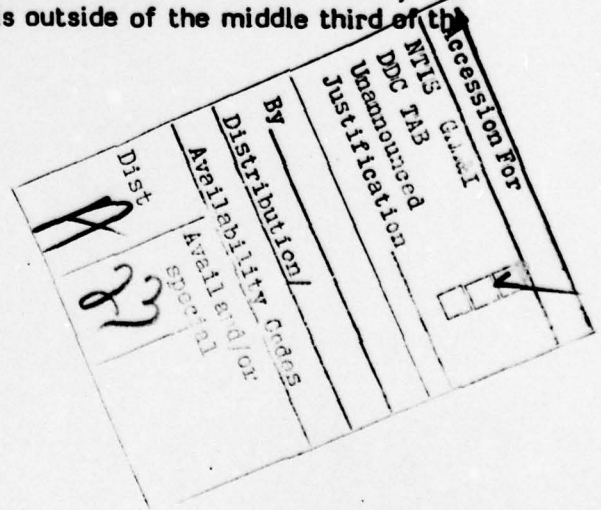
Name of Dam: Hinckley Dam

State Located: New York
County Located: Herkimer
Stream: West Canada Creek
Date of Inspection: June 6, 1978

ASSESSMENT OF GENERAL CONDITIONS

Areas of saturation and standing water on the embankments and at the toe, and fabric filter and stone fill remedial work are indicative of seepage problems. The discoloration of standing water at several locations could be evidence of fines migration. A boring program should be initiated immediately to determine the composition and in situ properties of the embankment materials to determine if they are satisfactory for the embankment as designed and constructed and to detect possible fines migration. Piezometers should be installed in the boreholes to evaluate pore pressure development throughout the embankment.

The masonry sections appear to be stable for all analyzed conditions, *except the Probable Maximum Flood (PMF). For the PMF, the* foundation reaction of the spillway is outside of the middle third of the base.



Concrete surfaces that have deteriorated due to spalling should be resurfaced to protect the concrete from further deterioration.

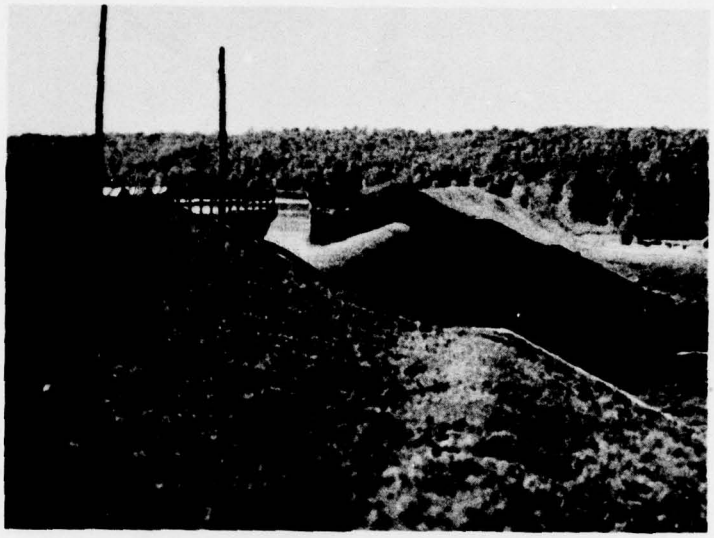
The spillway capacity is adequate for discharge of flood flows associated with the Probable Maximum Flood.

O'BRIEN & GERE ENGINEERS, INC.

John J. Williams, P.E.
Vice President

Approved by: _____
Clark H. Benn
Colonel, Corps of Engineers
District Engineer

Date: 19 September 1978



OVERALL VIEW OF DAM LOOKING SOUTH
FROM NORTH ABUTMENT

VIEW OF UPSTREAM FACE OF DAM

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PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
NAME OF DAM HINCKLEY DAM ID# NY 00181

SECTION I - PROJECT INFORMATION

1.1 GENERAL

a. Authority This report is authorized by the Dam Inspection Act, Public Law 92-367, and has been prepared in accordance with contract #1467-021 between O'Brien & Gere Engineers, Inc., and the New York State Department of Environmental Conservation.

b. Purpose of Inspection - The purpose of this inspection is to evaluate the structural and hydraulic conditions at Hinckley Dam and to determine if the dam constitutes a hazard to human life or property.

1.2 PROJECT DESCRIPTION

a. Description of Dam and Appurtenances (From drawings obtained from the New York State Department of Transportation). The Hinckley Dam is located on West Canada Creek, about one-half mile upstream from the town of Hinckley, New York. The structure consists of about 3,100 feet of earth embankment, a 400-foot long cyclopean masonry spillway with concrete wingwalls, and a cyclopean masonry non-overflow section about 65 feet long.

The earth embankment has a maximum structural height of about 48 feet, and is provided with a concrete core wall extending from 4 feet below the embankment crest into bedrock. The upstream slope of the embankment varies from 2.5:1 to 3.5:1 (horizontal to vertical), as shown on Figure 4. The downstream slope is 2:1, and is provided with two 8-foot wide benches.

The masonry non-overflow section is located to the north side of the spillway. The structure houses four 60 inch reservoir outlet pipes controlled by sluice gates at the upstream side of the structure. Operating assemblies for the sluice gates are located in a chamber at the top of the non-overflow section. The outlet pipes are also controlled by 48-inch spur- geared valves located in a gate chamber immediately downstream of the non-overflow section. Refer to Figure 5 for details of the outlet pipes, gates, and valves.

The south wingwall of the spillway is provided with sluice gates controlling discharge into a 42-inch cast iron pipe. The pipe is used to provide water for the City of Utica, New York.

The masonry spillway is an ungated ogee type section anchored with 2.5 inch diameter anchor bolts through the keyway at 8-foot centers for the length of the structure.

The dam is owned and operated by the New York State Department of Transportation (N.Y.S.D.O.T.), Waterways Maintenance Division. The reservoir provides flow augmentation for the Erie Canal, recreation, and water supply for the City of Utica, New York.

b. Size Classification - Hinckley Reservoir has a storage volume of 92,000 acre-feet at the normal pool elevation of 1225.0 feet above mean sea level (MSL). The height of the structure from the top of dam to the streambed elevation at the toe of the spillway is approximately 90 feet. The dam is in the large size category as defined by the Recommended Guidelines for Safety Inspection of Dams.

c. Hazard Classification - The town of Hinckley, New York is located along West Canada Creek within one-half mile of the dam. A failure of the dam could result in the loss of many lives and extensive economic losses. Therefore, the structure is in the high hazard category as defined by the Recommended Guidelines for Safety Inspection of Dams.

1.3 PERTINENT DATA (from information supplied by N.Y.S.D.O.T. and New York State Department of Environmental Conservation (N.Y.S.D.E.C.) or other sources as noted)

a. Drainage Area - The drainage area to Hinckley Reservoir is about 370 square miles, as determined by the use of United States Geological Survey quadrangle sheet (2 degree), for Utica, New York. The basin is completely within the Adirondack State Forest, and is well vegetated and undeveloped.

b. Discharges - The Hinckley Dam was designed for a maximum reservoir elevation of 1232.2 feet MSL. The corresponding spillway outflow is about 27,000 cubic feet per second (cfs). The spillway capacity with the reservoir water surface at the top of dam (elevation 1240.0) is about 80,000 cfs. The combined discharge capacity of the outlet pipes was calculated to be 650 cfs. According

to Mr. Russel Logalbo, Chief Engineer of the Utica Water Supply Board, the City of Utica is limited to a maximum withdrawal rate of 75 cfs for water supply.

c. Reservoir Data (from 15 minute quadrangle sheet for Hinckley, New York)

Normal Pool (Elevation 1225.0)

Length - 34,000 feet
Area - 3,070 acres
Volume - 92,000 acre-feet

Top of Dam (Elevation 1240.0)

Length - 40,000 feet
Area - 9,400 acres
Volume - 154,000 acre-feet

Maximum Pool (PMF)

Length - 38,000 feet
Area - 7,900 acres
Volume - 139,000 acre-feet

d. Dam Data

Embankment

Top elevation - 1242.0
Length - 3,100 feet
Height - 48 feet (maximum)
Top width - 11 feet
Slopes - See Figure 4
Impervious core - concrete core wall
Core wall height - 65 feet maximum
Zoning, cutoff, grout curtain - unknown

Non-Overflow Section

Top elevation - 1240.0 feet
Length - 65 feet
Height - 80 feet (maximum)
Type - cyclopean masonry

e. Spillway

Type - cyclopean masonry overflow
Length of weir - 400 feet
Crest elevation - 1,225 feet
Height - 83 feet (maximum)
Gates - none
Downstream channel - stilling basin and West Canada Creek

f. Engineering Data - The information available for review of the Hinckley Dam included:

1) A set of 36 drawings for the dam and appurtenant structures
2) Documents and inspection reports supplied by N.Y.S.D.E.C.
3) Discussion with Mr. Frank Jennings, Maintenance Engineer, N.Y.S.D.O.T., Utica, New York, concerning an area of stonefill and fabric filter remedial work observed on the north embankment. No drawings or reports pertaining to the remedial work were made available for review.

1.04 OPERATING AND MAINTENANCE PROCEDURES

a. Operation - Mr. Pritchard, gate operator at the dam, stated that operation was limited to regulating the gate openings of the outlet works for augmentation of flow in the Erie Canal. The City of Utica, New York, is allowed a maximum withdrawal rate from the reservoir of 75 cfs for water supply.

b. Maintenance of Dam and Operating Facilities - The mechanical appurtenances are old, but well maintained. The dam appears to receive limited maintenance, as evidenced by the general appearance of the earth embankments and gravity structures.

c. Flood Warning System - According to Mr. Pritchard and Mr. Jennings, no flood warning system has been established.

SECTION 2 - VISUAL INSPECTION

2.1 FINDINGS

a. General - The field inspection of the Hinckley Dam took place on June 6, 1978. At the time of inspection the reservoir water surface was about one-half foot below the spillway crest. Water was observed flowing from the outlet pipes. No underwater areas were inspected.

b. Dam - Riprap on the upstream face is composed of large angular stone varying in size from about 12 cubic feet to about 60 cubic feet. The riprap placement is very uniform and appears to have been placed on the slope, rather than dumped.

A depression was noted on the upstream slope of the north embankment. A localized area about 6 feet from the top of dam appears to have settled about 6 inches. The visible portion of the upstream slope was measured at about one horizontal to one vertical. Measurement of the upstream extension of the north wingwall revealed a 2.5 to 1 slope (horizontal to vertical). The available drawings indicate a 2.5 to 1 slope for both the wingwall and the upstream embankment slope. The slope of the upstream face of the south embankment was measured to be 2.5 to 1. The observed inconsistancy on the north embankment could indicate a widespread settlement in this area.

The downstream slope of the north embankment is provided with two benches. The benches are sloped towards the center of the embankment to provide for surface drainage. A swale is constructed near the center of the embankment. The swale extends from the upper bench to a ditch downstream of the toe of the embankment. Standpipes were noted on each bench about 30 feet from the north wingwall. The water levels below the upper and lower benches were measured at 24 feet and 11.8 feet respectively. Just above the toe of the embankment is an area of stone fill over a fabric filter extending about 20 feet up the embankment. The area extends laterally about 75 feet from the north wingwall, and is provided with a 6 inch corrugated metal underdrain pipe. The pipe terminates at a drainage ditch downstream of the toe. Flow from the pipe was clear and estimated at 1 gpm. See Figure 2 for the approximate location of the standpipes and stone fill.

Seepage was observed in the drainage ditch downstream of the north embankment toe. The seepage, which was brown or rust colored

and appeared oily, was entering at several locations on each side of the ditch. The sources of seepage could not be determined.

The upstream side of the wingwall is deeply spalled at the construction joints, and aggregate is exposed over much of the surface. Widespread surface spalling was noted on the vertical face of the downstream wingwall extension. Some minor seepage was observed along the wingwall adjacent to the stone fill area. Seepage was also evident about 10 feet south of the downstream end of the wingwall. The ground surface was saturated and some discoloration of the water was observed at this location. A retaining wall for deflecting flow from the outlet pipes is constructed downstream of the non-overflow section. Surface spalling of the concrete was observed at all construction joints. Weep holes were located several feet above the tailwater pool at approximately 30-foot intervals along the retaining wall. Flow from the weep holes was estimated at .5 cfs. This high flow may be the result of seepage that is flowing unimpeded from the embankment foundation or saturated areas and through the horizontal joints of the thinly bedded rock.

The concrete of the non-overflow section has undergone local surface spalling. Some age cracks were noted, but these appear to be surficial and should not affect the safety of the structure. Aggregate is exposed over approximately 60 per cent of the non-overflow masonry section of the downstream face. A concrete valve chamber extends from the base of the non-overflow section. The chamber houses 48 inch spur geared valves for the reservoir outlet pipes.

The spillway section appears to be in fair condition. Minor surface spalling and abrasive wear of the concrete surface were apparent, but in general the spillway concrete appeared to be sound.

The upstream face of the south embankment was in good condition. The slope of the visible portion of the embankment was measured as 2.5:1 (horizontal to vertical). Small trees and bushes growing on the south embankment have been recently cut down. Animal burrow holes were noted at many locations on the south embankment. The lower third of the south embankment near the spillway is very moist, with local saturated areas. The vegetation in this area is indicative of a prevailing moist or saturated condition.

The downstream slope is overgrown with tall grass, weeds, and brush. Trees and large brush have been cut down recently, and portions of the embankment appear to have been burned clear of vegetation.

Drainage of the downstream slope is accomplished by backsloping the two benches to form swales parallel to the crest. Cobblestone lined ditches, constructed at right angles to the benches, are provided at 300 feet intervals for drainage down the slope. A ditch parallel to the embankment is provided at the toe. Standing water was observed in the toe ditch, and in many areas standing water was widespread, creating swampy conditions. The standing water was brown or rust colored. The vegetation in these areas is indicative of a prevailing moist or saturated condition (See the bottom photograph on page A1).

c. Appurtenances - The masonry non-overflow section is provided with four 60 inch outlet pipes controlled by sluice gates at the upstream side of the structure. The 60 inch pipes are connected to 48 inch spur-geared valves by means of a reducer section. The valves are located in a gate chamber at the downstream side of the structure. Water was being discharged from two of the pipes at the time of inspection. The operating assemblies for the pipes are well lubricated and appeared to be in fair condition.

A gate chamber adjoins the south abutment of the spillway. This chamber is the intake for the water supply to the City of Utica. The chamber was locked and no inspection was made of these features.

SECTION 3 - HYDROLOGY AND HYDRAULICS

According to the Recommended Guidelines for Safety Inspection of Dams, the Spillway Design Flood is the Probable Maximum Flood (PMF). The PMF was calculated from the 24 hour Probable Maximum Precipitation (PMP) using standard reduction factors. The flood hydrograph was constructed from the Snyder unit hydrograph using average coefficients. Flood routing through the reservoir was performed assuming the gated outlets to be closed. The peak inflow and outflow rates were calculated as 73,900 cfs and 56,500 cfs respectively. The peak outflow corresponds to a stage of 11.6 feet above the spillway, or 3.4 feet below the top of dam. Therefore, the spillway is hydraulically adequate for the PMF.

A drawdown analysis was performed assuming discharge from four 60 inch pipes, the starting water surface at the spillway crest, and no inflow. According to the calculations, complete drawdown of the reservoir would take 85 days.

SECTION 4 - STRUCTURAL STABILITY

4.1 VISUAL OBSERVATIONS AND STABILITY ANALYSIS

Stability analyses were performed on both the spillway and the non-overflow masonry sections of the Hinckley Dam. More complete analyses would require information on foundation materials and the effects of side shear against adjoining structures.

Review of the stability analyses for the non-overflow section indicates that adequate factors of safety are present and that the foundation reaction is in the middle third of the base for all conditions analyzed.

Review of the stability analyses for the spillway indicates that satisfactory factors of safety are present for the normal pool and ice load conditions. For the PMF loading, the resultant of forces is outside the middle third of the base.

The remedial measures previously employed on the north embankment and noted during the field inspection visit, indicates that the embankment is susceptible to excessive seepage at this location. Saturation of portions of the embankment at other locations, and standing water near the toe of the embankments indicate that seepage could be widespread. The sources of seepage could not be determined during the inspection. Animal burrow holes and tree trunks in the south embankment are indicative of infrequent maintenance. The burrow holes and root systems of trees create potential seepage paths through the embankment. No information was made available on earth materials and construction techniques for the embankments. Assessment of the stability of the earth embankments is not within the scope of this report due to unknown conditions such as: the condition of the concrete core wall, the extent and sources of seepage, and the in situ properties of the embankment and foundation materials.

4.2 GEOLOGY AND SEISMIC STABILITY

The Reservoir is located within the western extremity of the Hudson-Mohawk physiographic province, between the Adirondack Highlands and the Tug Hill Upland. A northeast trending lineament is located to the north side of the north abutment. The lineament is an extension of a known fault zone. The bedrock underlying the dam is a

thinly bedded Ordovician limestone. Outcrops of this bedrock, noted downstream of the north abutment, were almost horizontally bedded. The dam is within Seismic Risk Zone 2 of the Seismic Zone Map of Contiguous States. The masonry sections of the dam were analyzed for an earthquake acceleration of .05g. Review of the stability analyses for both masonry sections indicates that the stability requirements of Chapter 4 of the Recommended Guidelines for Safety Inspection of Dams are met. Assessment of seismic stability of the earth embankments could not be made due to the unknown character of the embankment materials.

SECTION 5 - ASSESSMENT/REMEDIAL MEASURES

5.1 ASSESSMENT

The remedial work completed in November, 1977, the high flow from the weep holes in the downstream wingwall, and the areas of saturation and standing water on the embankments and at the downstream toe, indicate that excessive seepage is occurring. The seepage may be occurring from the embankment or the foundation. Discoloration of standing water, evident at several locations, could be indicative of fines migration or precipitation of ferrous impurities in the embankment or foundation. In addition, the observed depression on the upstream slope of the north abutment may be the result of fines migration through the embankment.

The masonry sections appear to be stable for all analyzed conditions, except the PMF. For the PMF loading, the foundation reaction of the spillway section is outside of the middle third of the base. Surface spalling has occurred on most concrete surfaces.

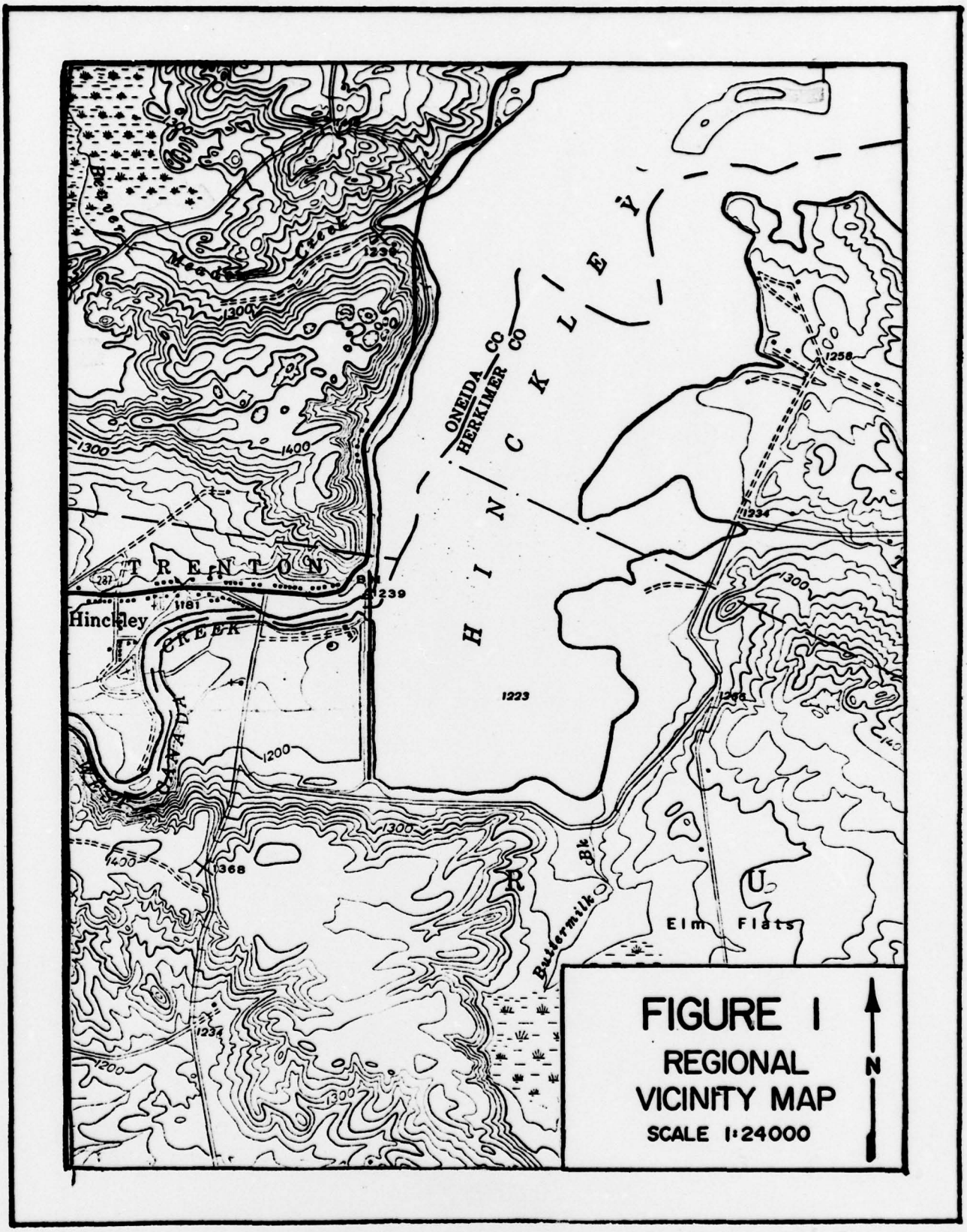
5.2 REMEDIAL MEASURES

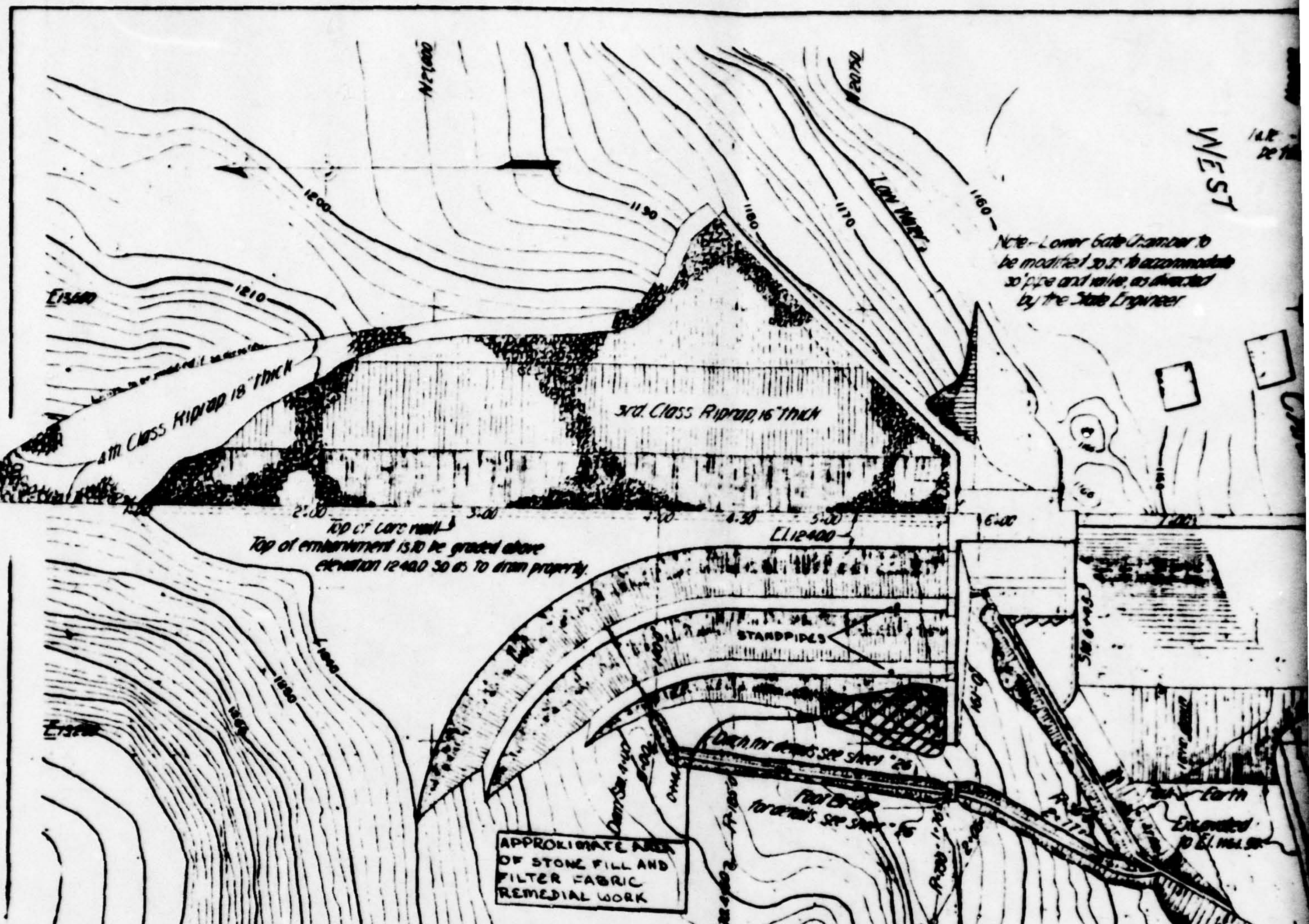
Concrete surfaces that have deteriorated due to spalling should be resurfaced to protect the underlying concrete from further spalling.

A boring program should be initiated immediately at several selected sections of the embankment to include, but not be limited to, determination of the composition and in situ properties of the embankment and foundation materials, to establish if they are satisfactory for the embankment core wall and foundation as designed and constructed, and to detect possible fines migration. Piezometers should be installed in the boreholes to evaluate pore pressure development throughout the embankment. The results of these investigations should be used to perform seepage and stability analyses for the embankments.

The earth embankments should be monitored regularly for signs of increased seepage and turbid water. The embankments should be mowed regularly to prevent the growth of deep rooted vegetation and to deter burrowing animals.

FIGURES


HINCKLEY
ONEIDA CO
HERKIMER CO
TRENTON
Hinckley
CREEK
Elm Flats
Buttermilk
1223
1300
1400
1200
1368
1239
1181
1258
1234
287
FIGURE 1
REGIONAL
VICINITY MAP
SCALE 1:24000
N


WEST
Note - Lower Gate Chamber to be modified so as to accommodate 30" pipe and valve, as directed by the State Engineer
Low Water
4th Class Riprap 18" thick
3rd Class Riprap 15" thick
Top of Core wall
Top of embankment is to be graded above elevation 1240.0 so as to drain properly.
El. 1240.0
STANDPIPES
Ditch for details see Sheet "25"
Foot Bridge for details see Sheet
APPROXIMATE AREA OF STONE FILL AND FILTER FABRIC REMEDIAL WORK
Earth
Excavated to El.
Existing
Note - Top of wall to be at El.1125 except where rock surface is below that elevation. Where rock surface is below El.1125.0, top of wall is to be at rock surface.
Original earth surface at wall.
Approx. rock
El. 1125.0
Low Water
Original earth surface
Approx. rock
Sectional Elevation A-A
Scale 1"=20'
Note. - Def. bars 6'-0" lg. to extend 4' into solid rock. Bars to be placed at corners of 6'x6' panels unless otherwise shown on the plans or directed by the State Engineer.
Section B-B
Section C-C
20 10 0 20 40 60 80 100
Scale 1"=20'
100 50 0
Scale
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TRACED BY C.G. Laughree
1st CHECK BY
2nd CHECK BY

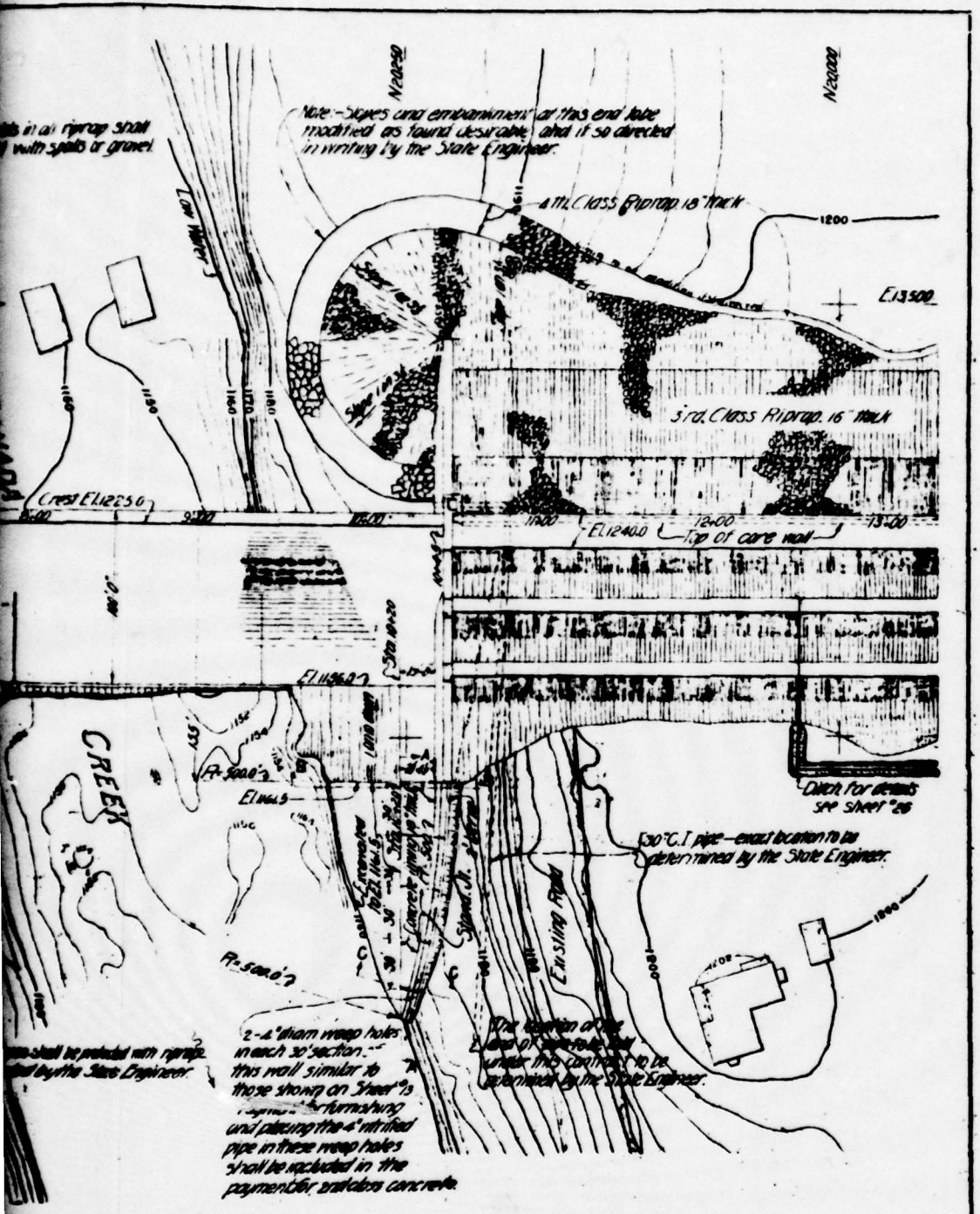


FIGURE 2
Contract No. 50.

Erie Canal **Section 5**

Reservoir and Dam at Hinckley.

GENERAL PLAN OF MASONRY DAM SHOWING FINISHED STRUCTURES

Scales as indicated

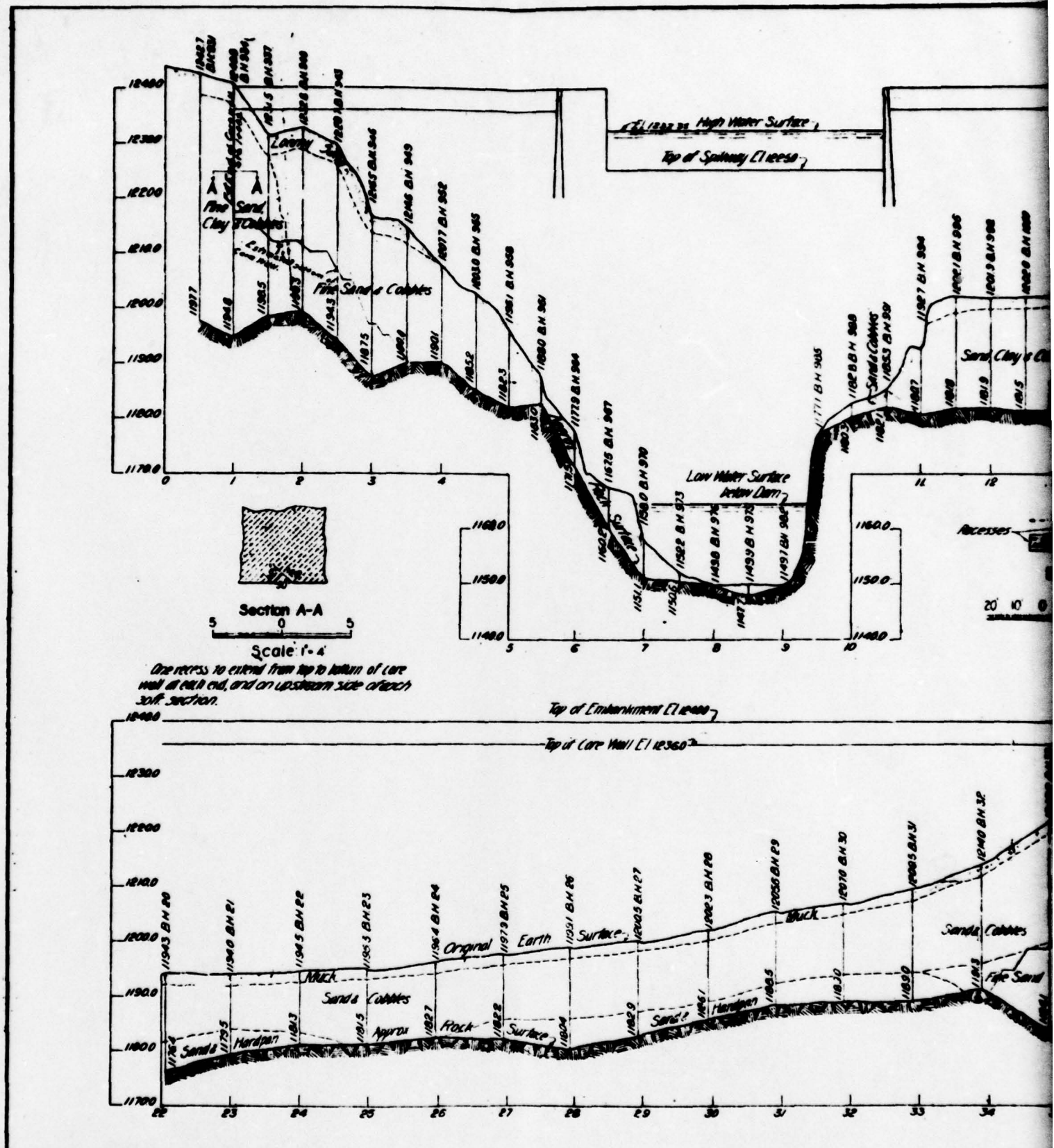
Examined and approved

Special Deputy State Engineer

Dec 27 1904

8

2



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TRACED BY C.G. Lampkin
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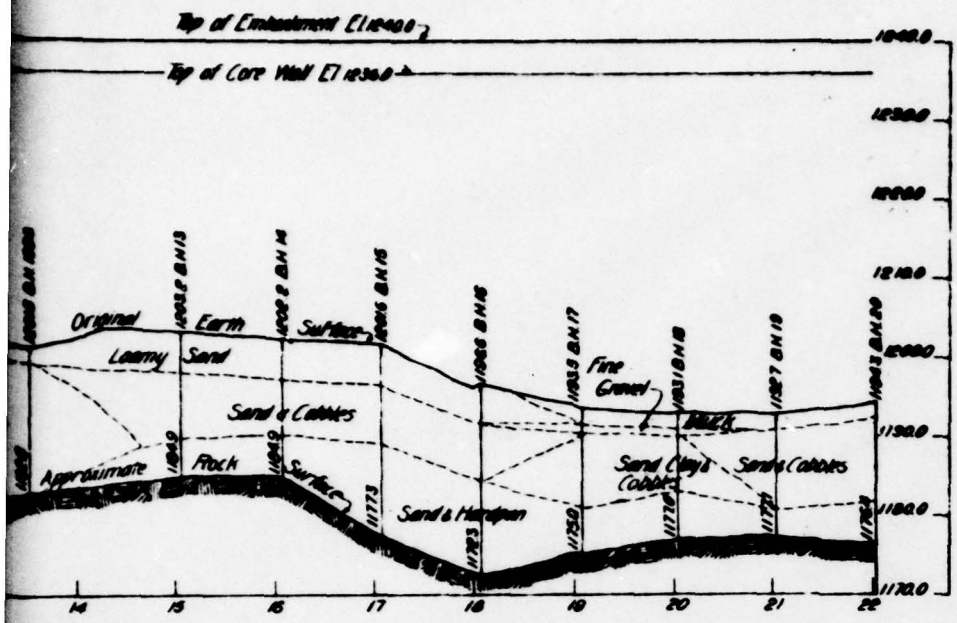
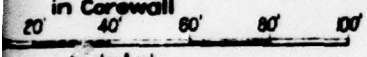
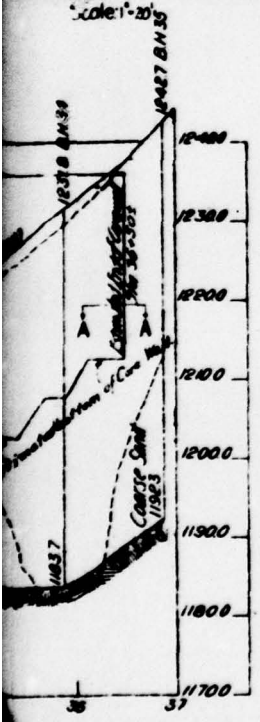
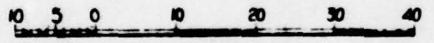


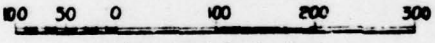
Plan showing Recesses in Carewall



Scale 1"-20'







Note - A thin layer of clay and sand from about 12' to 15' below the surface between stations 19-00 and 30-00 was shown by some dry samples in the vicinity of this line.

Note -
The bases of structures shown on any of the plans of this contract shall be considered as approximate only, and may be ordered by the State Engineer in writing to be of any elevation and of any dimensions necessary to give a proper foundation.

The ends of the corewall shall be extended into the hill to the distance deemed necessary by the Engineer to give proper security. All work connected with such extension shall be paid for at corresponding contract prices.

Contract No. 50.

Erie Canal **Section 5**

Reservoir and Dam at Hinckley.

PROFILE ON AXIS OF DAM.

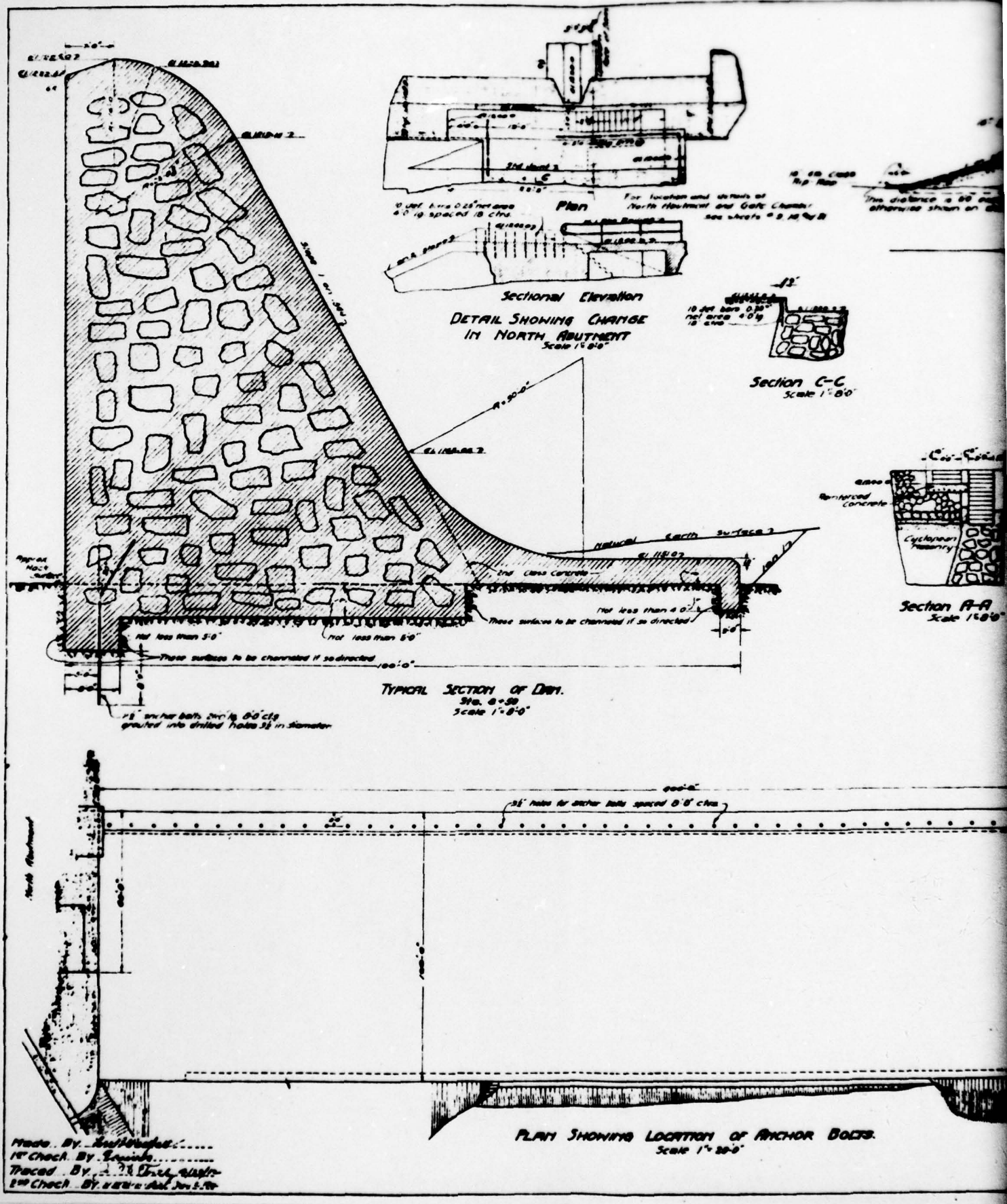
Scales as indicated

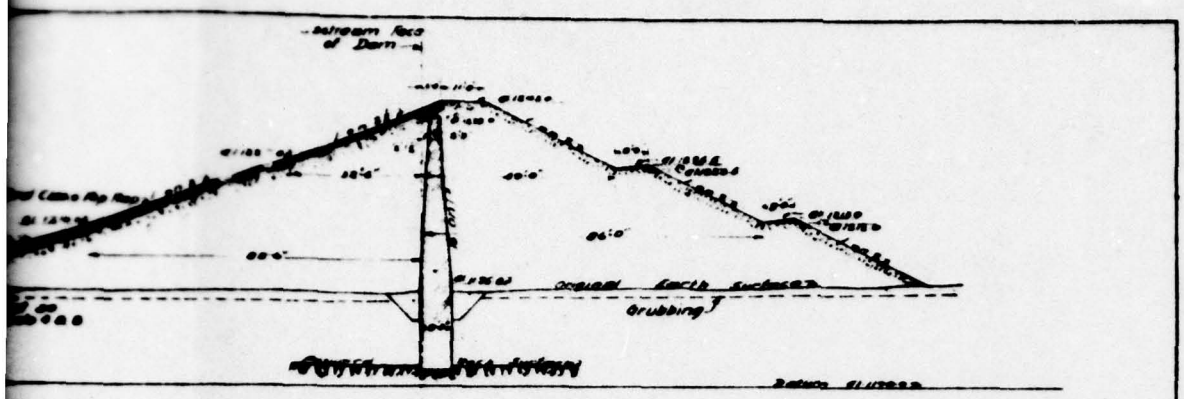
Recommended and approved

FIGURE 3

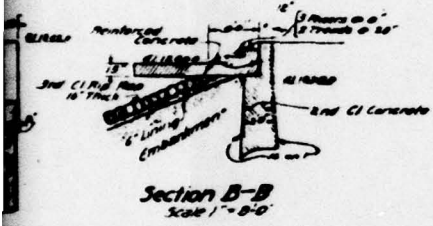
6

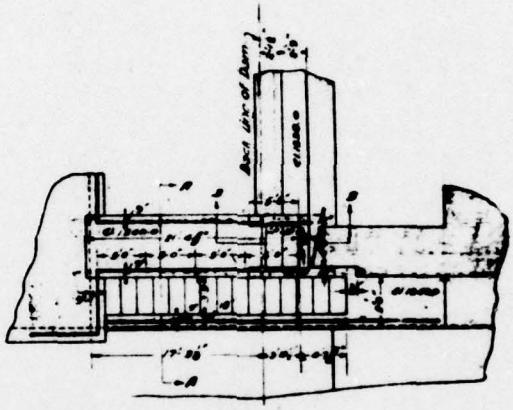
2

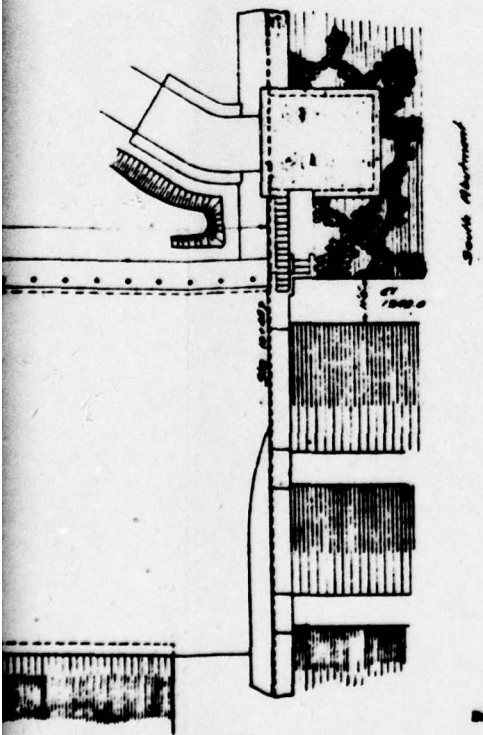

Plan
Sectional Elevation
DETAIL SHOWING CHANGE IN NORTH ABUTMENT
Scale 1"=8'0"
Section C-C
Scale 1"=8'0"
Section A-A
Scale 1"=8'0"
Natural Earth Surface
1st Class Concrete
Cyclopean Masonry
Reinforced Concrete
These surfaces to be channeled if so directed
Not less than 5'0"
Not less than 6'0"
Not less than 4'0"
100'0"
TYPICAL SECTION OF DAM.
Sta. 0+50
Scale 1"=8'0"
PLAN SHOWING LOCATION OF ANCHOR BOLTS.
Scale 1"=20'0"
Made By
1st Check By
Traced By
2nd Check By

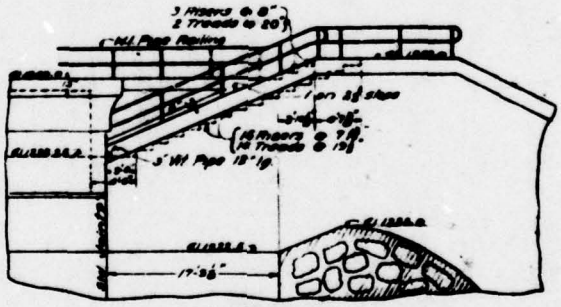


EARTH SECTION
Sta 19+00
Scale 1"=20'-0"









PLAN SHOWING CHANGE IN SOUTH ABUTMENT
Scale 1"=5'-0"

FIGURE 4

Contract No. 50.

ALTERATION NO 2

Erie Canal — Section 5

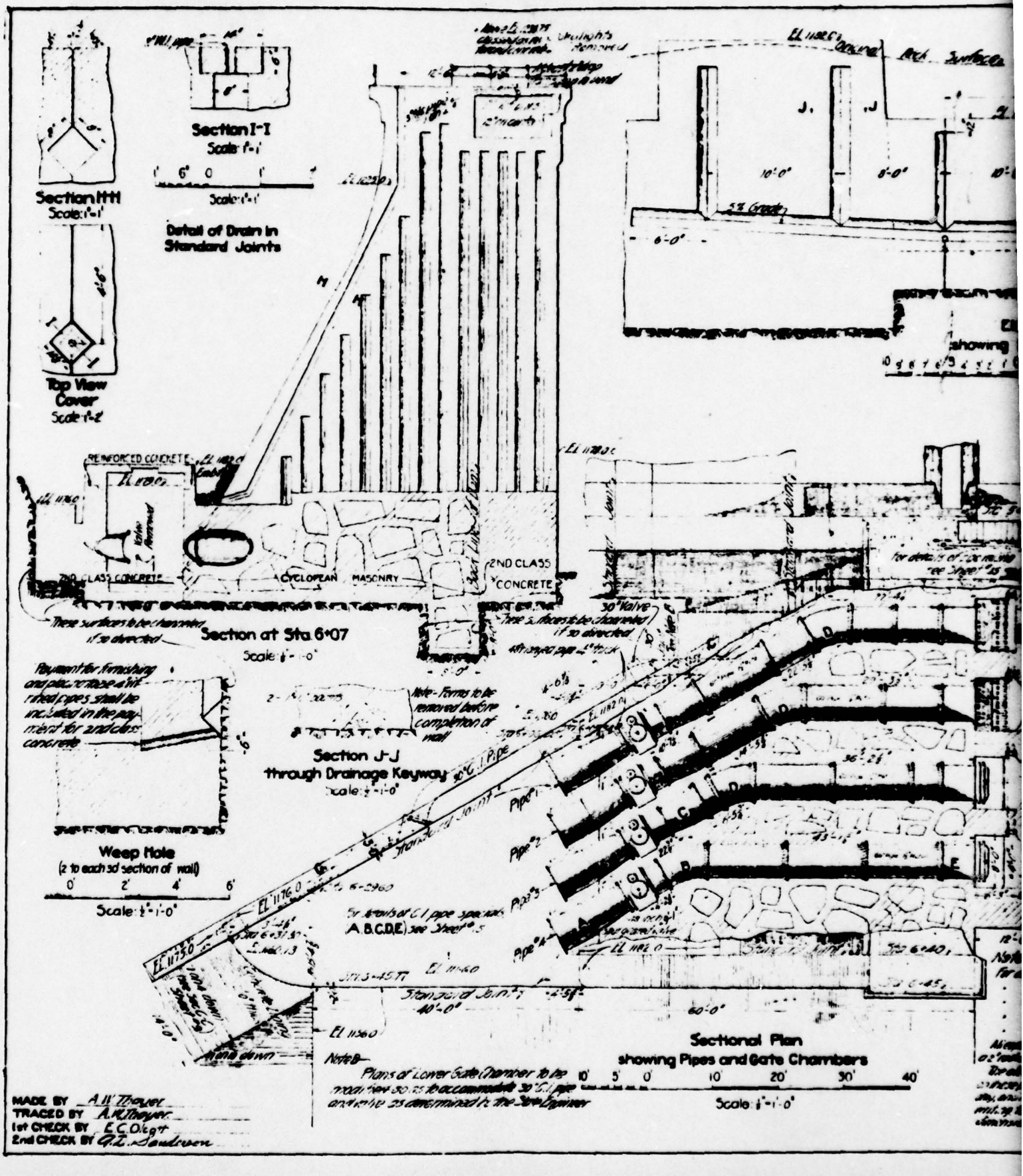
REVISED DETAILS OF MASONRY & EARTH SECTIONS OF HINCKLEY DAM

Scales as indicated

Examined and approved
May 2 1912
Chief Engineer

Examined and approved
May 6 1912
Special Deputy State Engineer

24.


Section I-I
Scale 1"-1'
Detail of Drain in Standard Joints
Section HH
Scale 1"-1'
Top View Cover
Scale 1"-2'
Section at Sta 6+07
Scale 1/4"-1'-0"
Section J-J through Drainage Keyway
Scale 1/2"-1'-0"
Weep Hole
(2 to each sq section of wall)
Scale 1/2"-1'-0"
Sectional Plan
showing Pipes and Gate Chambers
Scale 1/8"-1'-0"
REINFORCED CONCRETE
CYCLOPEAN MASONRY
2ND CLASS CONCRETE
30"Valve
Pipe 1
Pipe 2
Pipe 3
Pipe 4
Note B
MADE BY A.W.Thayer
TRACED BY A.K.Thayer
1st CHECK BY E.C.Otgt
2nd CHECK BY G.L.Sandvren

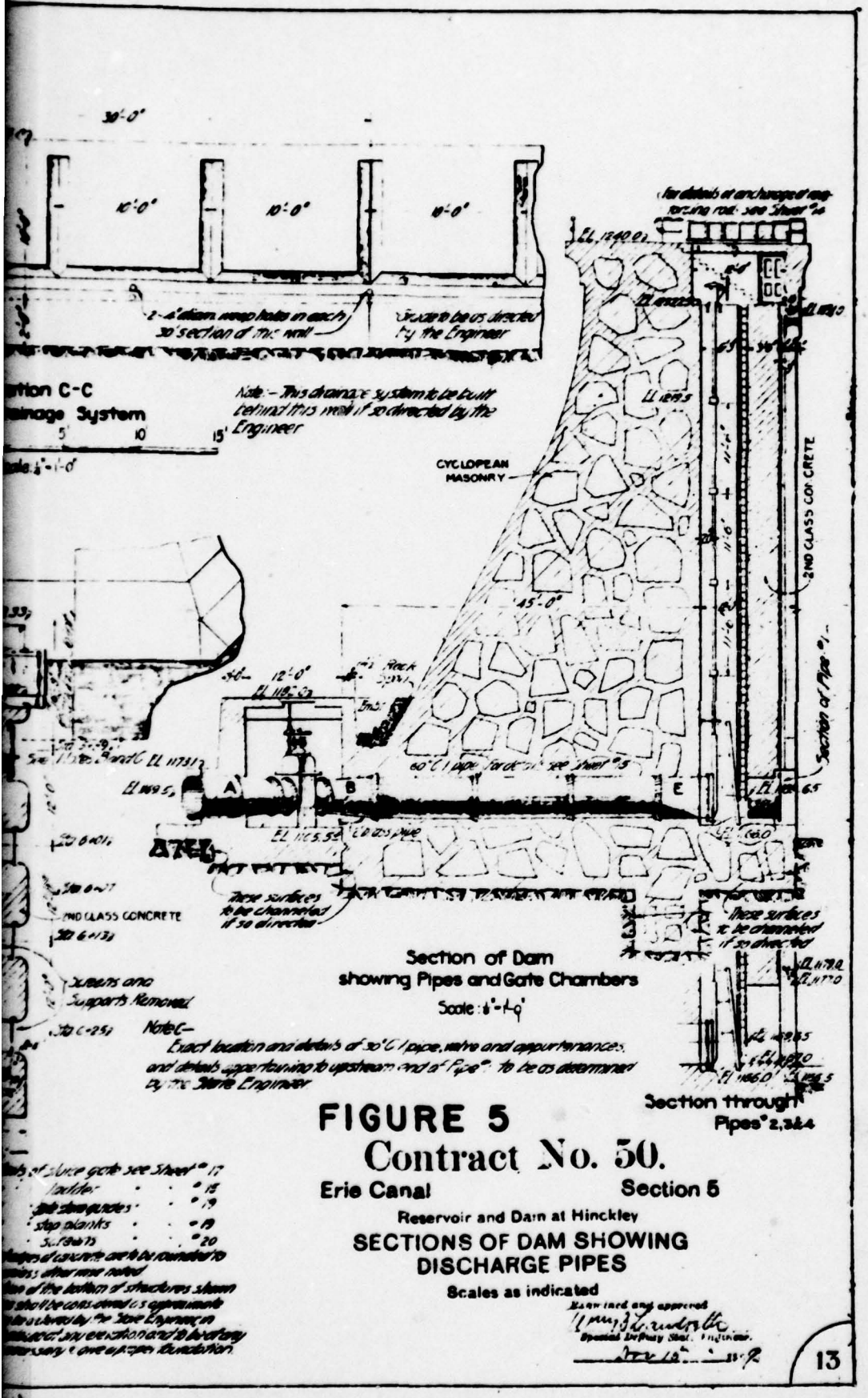


FIGURE 5

Contract No. 50.

Erie Canal **Section 5**

Reservoir and Dam at Hinckley

SECTIONS OF DAM SHOWING DISCHARGE PIPES

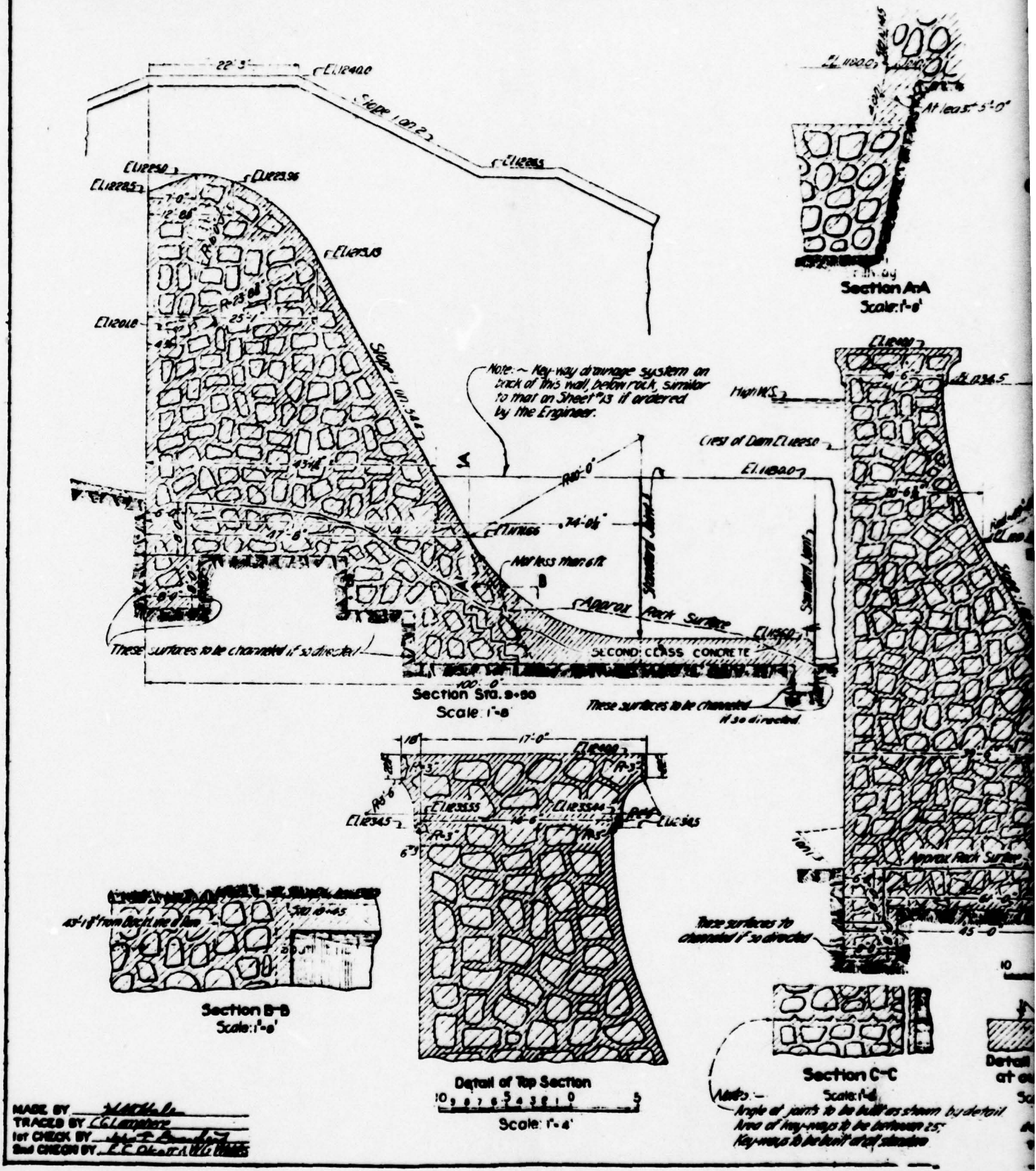
Scales as indicated

Examined and approved

Deputy State Engineer.

June 12th 1911

2



MADE BY M.A.Hale
TRACED BY C.C.Lampkin
1st CHECK BY
2nd CHECK BY

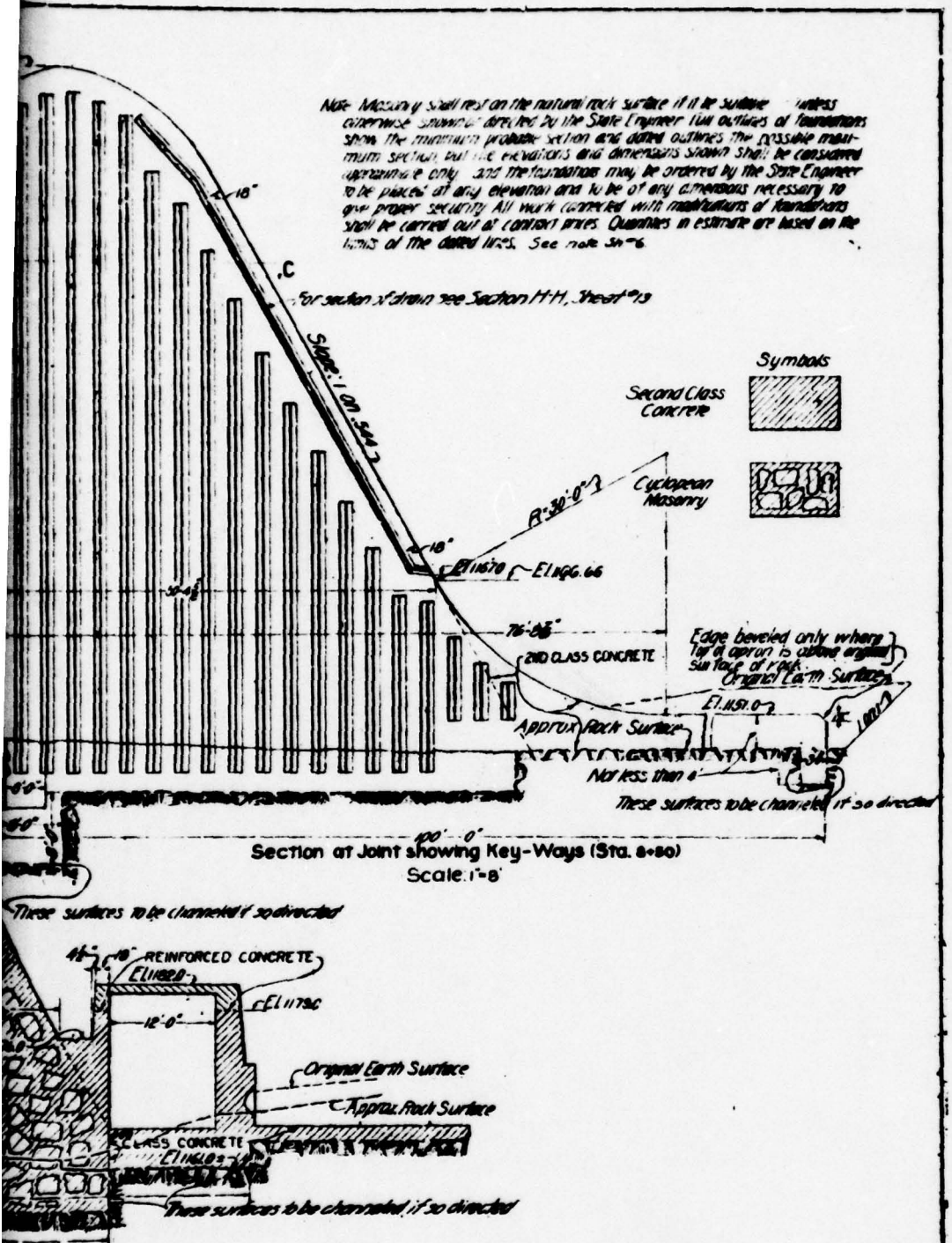


FIGURE 6
Contract No. 50.

Erie Canal **Section 5**

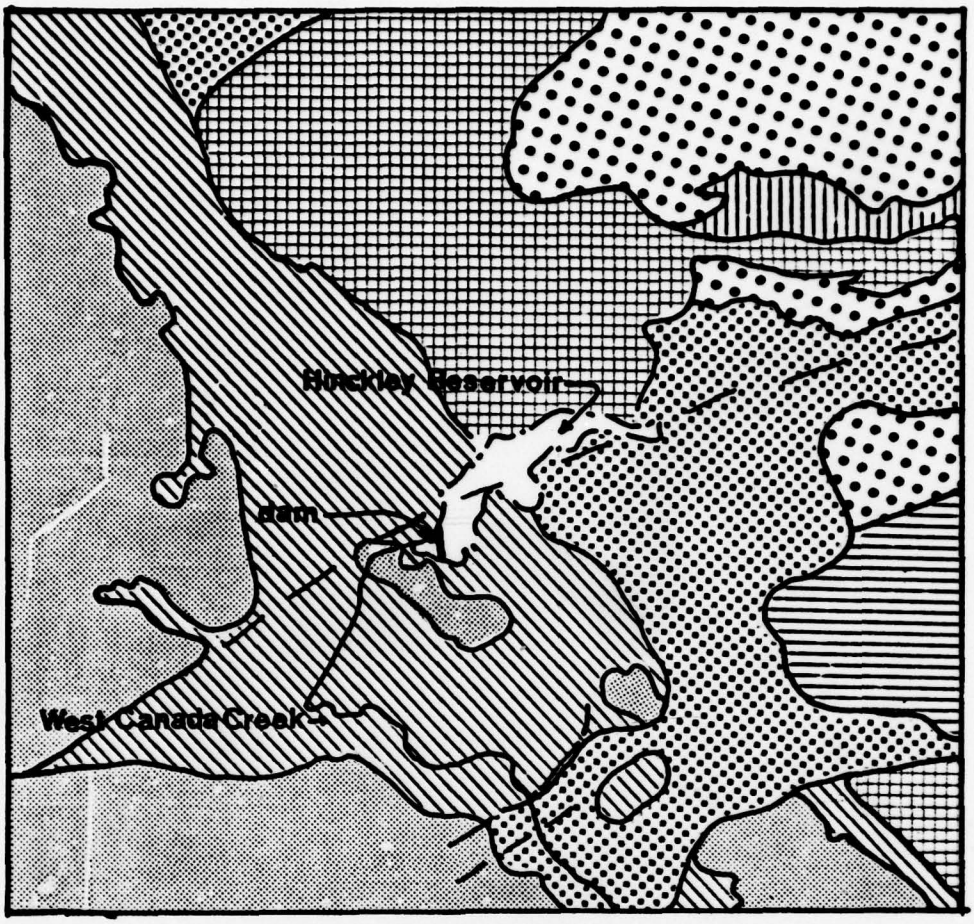
Reservoir and Dam at Hinckley.

DETAIL SECTIONS OF MASONRY DAM

Scales as indicated

Examined and approved
Deputy State Engineer

12



— — — Topographic Lineament

mug — Interlayered metasedimentary rock, granite and gneiss

Q — Glacial and alluvial deposits

hbg — Biotite and/or hornblende granitic gneiss

Ot — Limestones

bqpq — Biotite-quartz-plagioclase gneiss

Ou — Utica shale

mu — Undivided metasedimentary rock

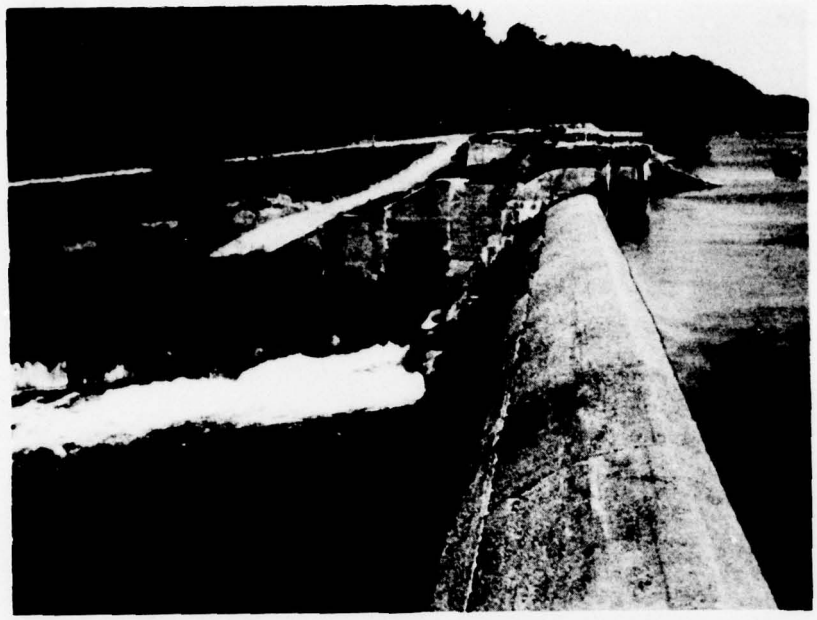
FIGURE 7

GEOLOGIC MAP

Scale 1:250000

APPENDIX

PHOTOGRAPHS



NORTH ABUTMENT, OUTLET WORKS, AND SPILLWAY

SATURATED AREA WITH STANDING WATER AT TOE OF EMBANKMENT

CRACKING AND SPALLING OF CONCRETE ON SPILLWAY WING WALL

VIEW OF DOWNSTREAM CHANNEL

FIELD INSPECTION REPORT

Check List
Visual Inspection
Phase 1

Name Dam Hinckley Reservoir Dam County Herkimer State New York Coordinators _____

Date(s) Inspection 6/6/78 Weather Clear Temperature 70°

Pool Elevation at Time of Inspection 1,224 M.S.L. Tailwater at Time of Inspection 1,170 M.S.L.

A3

Inspection Personnel:

Mr. George Elias Mr. James Ryan _____

Mr. Steven Snider _____ _____

Mr. David Campbell

Mr. David Campbell Recorder

Accompanied by:

Mr. Pritchard, gate operator, New York State Department of Transportation.

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBERSVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS CONCRETE SURFACES	Widespread spalling and surficial cracking of concrete structure was noted.	Cracks and spalled areas should be resurfaced to protect the underlying concrete.
STRUCTURAL CRACKING	None Noted.	None.
VERTICAL AND HORIZONTAL ALIGNMENT	No problems noted.	None.
MONOLITH JOINTS	No problems noted.	None.
CONSTRUCTION JOINTS	Some separation was observed at construction joints on portions of the wingwalls and the retaining wall in contact with water surfaces.	The separation should be monitored to detect any further movement.

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SEE PAGE ON LEAKAGE		
STRUCTURE TO ABUTMENT/EMBANKMENT JUNCTIONS	No problems noted.	None.
DRAINS	Weepholes were observed in the retaining wall downstream of the gravity section outlet works. The weep holes are spaced at about 30 feet and were noted to be discharging clear water at a rate of about 1 cfs each.	None.
WATER PASSAGES	The gravity section is provided with four 60 inch cast iron pipes controlled by sluice gates at the upstream side and 48 inch spur geared valves in the downstream gate chamber.	None.
FOUNDATION	Outcrops of flatly bedded limestone were noted along the north embankment of West Canada Creek downstream of the dam.	None.

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	None noted.	None.
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None noted.	None.
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	Remedial slope protection of filter fabric and stone on north embankment. Could not determine if the remedial work was due to a slope failure, excessive seepage, or for some other reason.	See Figure 2 for area of remedial work.
VERTICAL AND HORIZONTAL ALINEMENT OF THE CREST	No problems noted.	None.
RIPRAP FAILURES	None.	None.

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	No problems noted.	None.
ANY NOTICEABLE SEEPAGE	Minor seepage was noted at the toe of both earth embankments. At most locations, the seepage water was observed to have a rust or brown discoloration.	See comments on previous page.
STAFF GAGE AND RECORDER	None noted.	None.
DRAINS	Weep holes were observed in the retaining wall downstream of the outlet pipes at 30 foot intervals. Flow from each weep hole was estimated @.5cfs.	None.

UNGATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	The 400-foot concrete gravity spillway appears to be in fair condition. Some minor spalling and abrasive wear has occurred on the surface concrete.	None.
APPROACH CHANNEL	None.	None.
DISCHARGE CHANNEL	A concrete apron extends downstream from the spillway.	None.
BRIDGE AND PIERS	None.	None.

AB

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	Some surface spalling has occurred on the concrete surfaces of the gate chamber.	None.
INTAKE STRUCTURE	The intake structure is the non-overflow gravity Section. Sluice gates control inflow to the outlet pipes.	None.
OUTLET STRUCTURE	A concrete gate chamber is located just downstream of the non-overflow section. Discharge is controlled by 48 inch spur-geared valves at the outlet.	None.
OUTLET CHANNEL	A concrete apron is constructed downstream of the outlet and a retaining wall deflects flow towards the main channel.	None.
EMERGENCY GATE	None observed.	None.

INSTRUMENTATION

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	A survey monument was noted at the top of the non-overflow gravity section. The elevation was marked as 1239.9 feet MSL.	None.
OBSERVATION WELLS	None noted.	None.
WEIRS	None noted.	None.
PIEZOMETERS	Stand pipes were noted at each bench above the area of filter fabric and stone fill.	Water surface elevations were determined by splash to be 1202.0 feet MSL and 1201.7 feet MSL for the upper and lower benches respectively.
OTHER		

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Reservoir slopes are mild and well covered with vegetation.	None.
SEDIMENTATION	No estimate of reservoir sedimentation could be made.	None.

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	A truss bridge is constructed across West Canada Creek about 1,500 feet downstream of the spillway. The bridge creates only a mild obstruction.	None.
SLOPES	Downstream slopes are mild and well vegetated.	None.
APPROXIMATE NO. OF HOMES AND POPULATION	The town of Hinckley, New York, is located about one-half mile downstream of the dam. About 60 homes (250 people) are located within one mile of the dam.	None.

HYDROLOGIC AND HYDRAULIC COMPUTATIONS

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 1 OF
DATE 5/31/78
COMP. BY DBC
CHECKED BY BEH

NAME OF CLIENT Corps of Engineers
PROJECT Hinckley Reservoir

DRAINAGE AREA (BY PLANIMETER) ≈ 370 SQ. MI.

L = 50 MILES L_{CA} = 15 MILES

AVERAGE SNYDER COEFFICIENTS

C_p = .625 C_T = 2.0

$t_p = C_T(L \times L_{CA})^{.3} = 2.0(50 \times 15)^{.3} = 15.4$ HOURS

$t_r = t_p/5.5 = 2.8$ HOURS USE t_R = 3.0 HOURS

$t_{PR} = t_p + .25(t_r - t_R) = 15.4 + 25(3.0 - 2.8) = 15.45$

6 HOUR PMP = 21"

REDUCTION DUE TO PROBABLE MISFIT OF RAIN WITH STORM ISOHYETALS IS 10%

6 HR. PMP = 18.9"

DEPTH AREA - DURATION FOR PMP
(ZONE 1)

6 HR. PMP = 18.9" × .6 = 11.3"

12 HR. PMP = 18.9" × .74 = 14.0" 14.0 - 11.3 = 2.7"

24 HR. PMP = 18.9" × .83 = 15.7" 15.7 - 14.0 = 1.7"

CN=55 MOUNTAIN. FOREST - (LIGHTLY UNDEVELOPED)

TYP RAINFALL

TIME (HRS)		RAINFALL	
0-3	(5)	2.5" (75% 6HR. PMP)	
3-6	(4)	2.8" (72% 6HR. PMP)	
6-9	(6)	1.4"	12 HR. PMP - 6 HR. PMP
9-12	(3)	1.3"	
12-15	(7)	.5"	
15-18	(8)	.4"	24 HR. PMP - 12 HR. PMP
18-21	(2)	.4"	
21-24	(1)	.4"	

THIRD QUARTILE DISTRIBUTION

TIME (HRS)	RAINFALL
0-3	.4"
3-6	.4"
6-9	1.3"
9-12	2.8"
12-15	8.5
15-18	1.4"
18-21	.5"
21-24	.4"

RAINFALL-RUNOFF RELATION

ADJUSTED PMP INCREMENTS	Σ PMP	RUNOFF INCR.	RUNOFF Σ	LOSSES INCR.	LOSSES Σ
.4"	.4"	.0"	.0"	.4"	.4"
.4"	.8"	.0"	.0"	.4"	.8"
1.3"	2.1"	.0"	0"	1.3"	2.1"
2.8"	4.9"	.9"	.9"	1.9"	4.0
8.5"	13.4"	6.0"	6.9"	2.5"	6.5
1.4"	14.8"	.5"	7.7"	.6"*	7.1
.5"	15.3"	.0"	7.7"	.5"*	7.6
.4"	15.7"	.0"	7.7"	.4"*	8.0

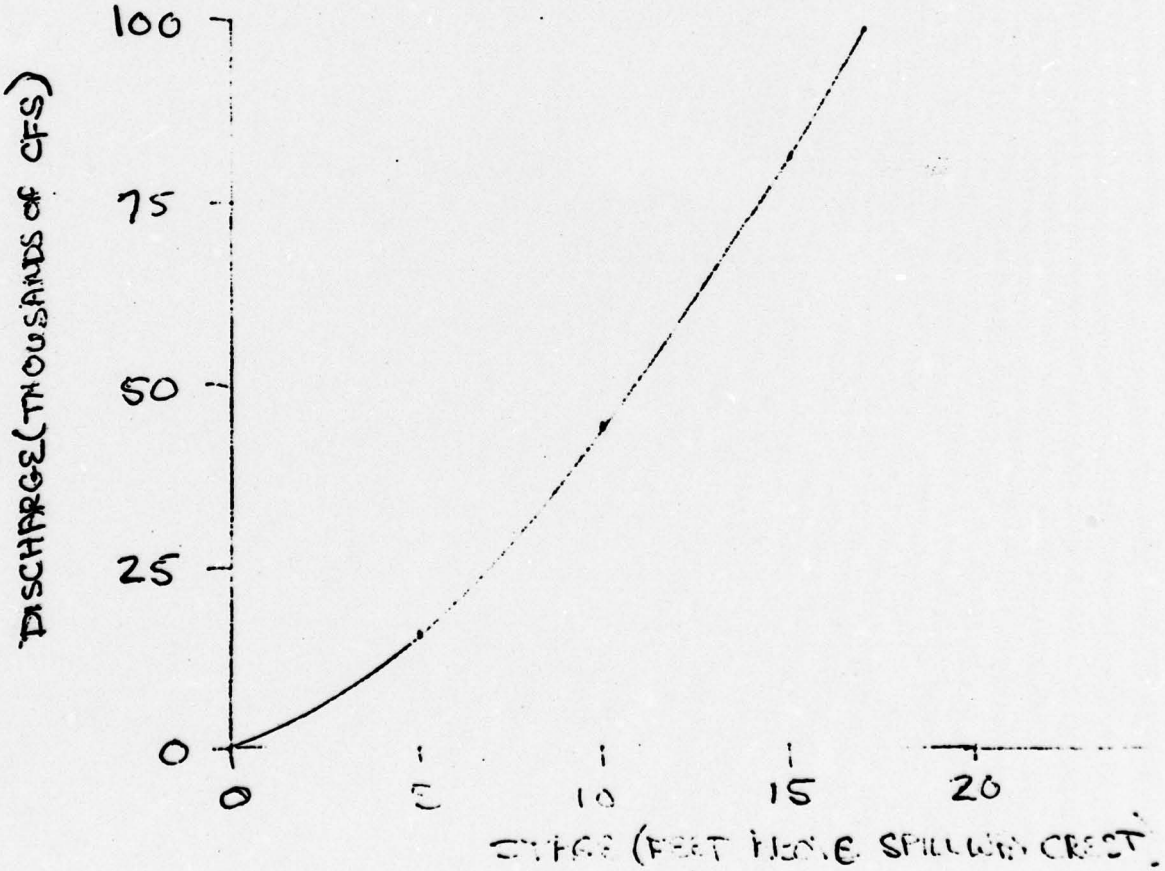
* MINIMUM LOSS RATE OF .2 IN/HR. = .6 IN/3 HR.

STAGE-DISCHARGE RELATION

$C = 3.5$

$Q = CLH^{3/2} = 3.5 \times 400 \times H^{3/2}$

$Q = 1400 H^{3/2}$



STAGE - STORAGE RELATION

BY PLANIMETER FROM (USGS QUAD)

ELEVATION - 1223' AREA - 2863 ACRES

ELEVATION - 1240' AREA - 4618 ACRES

ASSUME A LINEAR VARIATION IN SURFACE AREA FROM ELEV. 1223 TO 1240

(S) STAGE = 0 AT EL. 1225

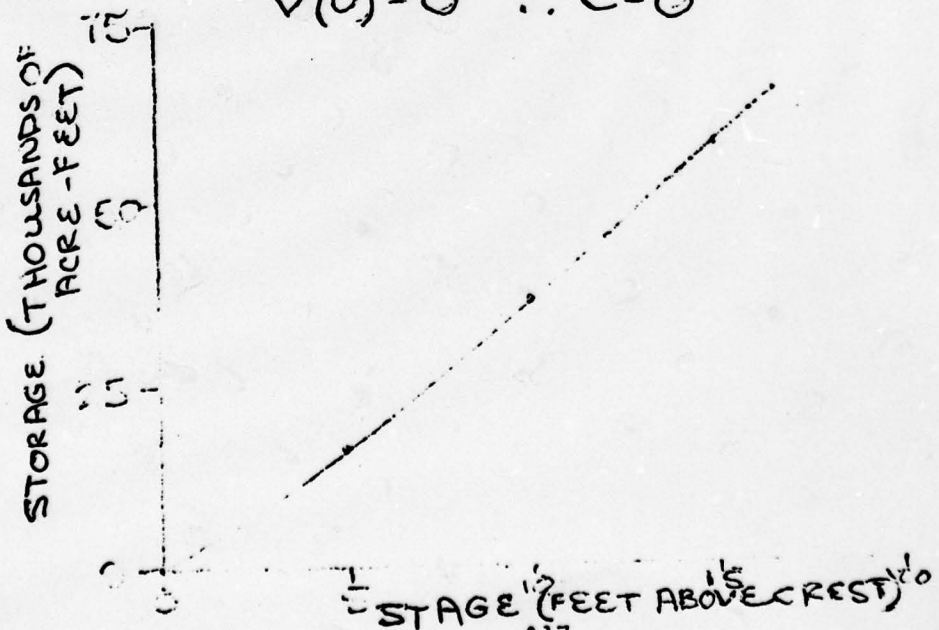
$A(-2) = 2863$

$A(15) = 4618$

$\therefore A = 103.25 + 3070$

$$V = \int (103.25 + 3070)\,dS = 51.65^{2} + 3070S + C$$

$V(0) = 0 \quad \therefore C = 0$



NAME OF CLIENT CORPS OF ENGINEERS

PROJECT HINCKLEY RESERVOIR

STORAGE (ACRE-FEET)	DISCHARGE (CFS)
0	0
6350	3960
13110	11200
20280	20580
27860	31600
35860	44270
44270	58200
53090	73340
62330	89600
67100	98130

Discharge capacity for outlet pipes.

$$H = \frac{V^2}{2g} + K_e\frac{V^2}{2g} + K_b\frac{V^2}{2g} + K_v\frac{V^2}{2g} + h_f$$

H = available head

K_e = entrance loss coefficient

K_b = bend loss coefficient

K_v = valve loss coefficient

$$h_f = \frac{29n^2L}{R^{1.33}}\left(\frac{V^2}{2g}\right) \quad \& \quad \frac{V^2}{2g} = \frac{Q^2}{2gA^2}$$

$K_e = .2 \quad K_b = .2 \quad K_v = 1.0 \quad n = .01$

$$H = \left(1 + .2 + .2 + 1 + \frac{29\times(.01)^2\times 90}{(1.25)^{1.33}}\right)\frac{Q^2}{2g\times\pi\times(2.5)^2}$$

$H = .0020\,Q^2 \qquad Q = 22.\,H^{1/2}$ (each pipe)

Outlet capacity at normal pool.

$$Q_{TOT} = 4\times 22(55)^{1/2} = 650\ cfs$$

H	50	40	30	20	10
Q	622	557	482	394	278

SURFACE AREA (NORMAL POOL) = 307.0 ACRES

ASSUME A LINEAR VARIATION IN SURFACE AREA FROM 307.0 ACRES AT THE CREST TO ZERO AT ELEVATION 1170.

ΔH (feet)	H AVE (feet)	Q AVE (cfs)	NET LVE (acres)	Δt (days)	t (days)
	50	622	2791	23	33
10	40	557	2233	20	43
10	30	482	1675	18	61
10	20	394	1117	14	75
10	10	278	558	10	85

```
**********
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
**********
```

NATIONAL DAM INSPECTION PROGRAM
HINCKLEY RESERVOIR
PMF HYDROGRAPH

JOB SPECIFICATION

NQ	NHR	NMIN	IDAY	IHR	IMIN	METRC	IPLT	IPRT	NSTAN
32	3	0	1	0	0	0	0	2	0

JOPER	NWT
3	0

********** ********** ********** ********** **********

SUB-AREA RUNOFF COMPUTATION

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
1	0	0	0	0	0	0

HYDROGRAPH DATA

IHYDG	IUNG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
0	1	370.00	0.00	0.00	0.00	0.000	0	0	0

PRECIP DATA

NP	STORM	DAJ	DAK
3	0.30	0.00	0.00

PRECIP PATTERN

.30	6.00	.80

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0.00	0.00	1.00	0.00	0.00	1.00	0.00	0.00	0.00	0.00

UNIT HYDROGRAPH DATA
TP= 15.45 CP= .63 NTA= 0

RECESSION DATA
STRTQ= 0.00 QRCSN= 0.00 RTIOR= 1.00

APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC= 6.05 AND R= 4.56 INTERVALS

UNIT HYDROGRAPH 28 END-OF-PERIOD ORDINATES, LAG= 15.30 HOURS, CP= .63 VOL= 1.00

747.	2713.	5311.	7817.	9647.	9763.	8636.	6939.	5567.	4467.
3545.	2876.	2308.	1852.	1486.	1193.	957.	768.	616.	494.
397.	318.	255.	205.	164.	132.	106.	85.		

END-OF-PERIOD FLOW

	TIME		RAIN	EXCS	COMP Q
1	3	0	.30	.00	672.
1	6	0	6.00	5.00	6034.
1	9	0	.80	.80	21651.
1	12	0	0.00	0.00	41074.
1	15	0	0.00	0.00	59152.
1	18	0	0.00	0.00	71721.
1	21	0	0.00	0.00	77108.
2	0	0	0.00	0.00	75971.
2	3	0	0.00	0.00	69468.
2	6	0	0.00	0.00	62075.

2	9	0	0.00	0.00	34484.
2	12	0	0.00	0.00	27671.
2	15	0	0.00	0.00	22204.
2	18	0	0.00	0.00	17817.
2	21	0	0.00	0.00	14297.
3	0	0	0.00	0.00	11477.
3	3	0	0.00	0.00	9209.
3	6	0	0.00	0.00	7392.
3	9	0	0.00	0.00	5927.
3	12	0	0.00	0.00	4756.
3	15	0	0.00	0.00	3816.
3	18	0	0.00	0.00	3062.
3	21	0	0.00	0.00	2457.
4	0	0	0.00	0.00	1972.
4	3	0	0.00	0.00	1582.
4	6	0	0.00	0.00	1270.
4	9	0	0.00	0.00	1019.
4	12	0	0.00	0.00	817.
4	15	0	0.00	0.00	595.
4	18	0	0.00	0.00	61.
4	21	0	0.00	0.00	0.
5	0	0	0.00	0.00	0.
		SUM	7.70	7.70	609878.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	73909.	72815.	55404.	25227.	609878.
INCHES		1.83	5.57	7.61	7.67
AC-FT		36125.	109950.	150186.	151287.

********** ********** ********** ********** **********

HYDROGRAPH ROUTING

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
2	1	0	9	2	0	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.000	0.00	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
0	0	0	0.000	0.000	0.000	-1.

STORAGE=	0.	6350.	13110.	20240.	27960.	35860.	44270.	53030.	62330.	67100.
OUTFLOW=	0.	3950.	11200.	20580.	31640.	44270.	58200.	73340.	89600.	99130.

	TIME			EOP STOR	AVG IN	EOP OUT
1	3	0		1078.	672.	672.
1	6	0		1749.	3798.	1121.
1	9	0		4828.	14289.	3011.
1	12	0		11104.	31364.	9056.
1	15	0		19971.	50361.	20176.
1	18	0		29507.	65687.	34272.
1	21	0		37489.	72815.	46063.
2	0	0		42284.	69991.	54778.
2	3	0		43219.	59711.	56659.
2	6	0		41533.	44961.	53666.
2	9	0		38460.	34729.	49577.
2	12	0		34952.	31977.	45584.
2	15	0		31170.	29917.	41490.
2	18	0		27666.	28010.	37336.
2	21	0		24467.	26057.	33283.
3	0	0		21552.	23984.	29663.
3	3	0		18911.	21831.	26334.
3	6	0		16513.	19706.	23336.

A22

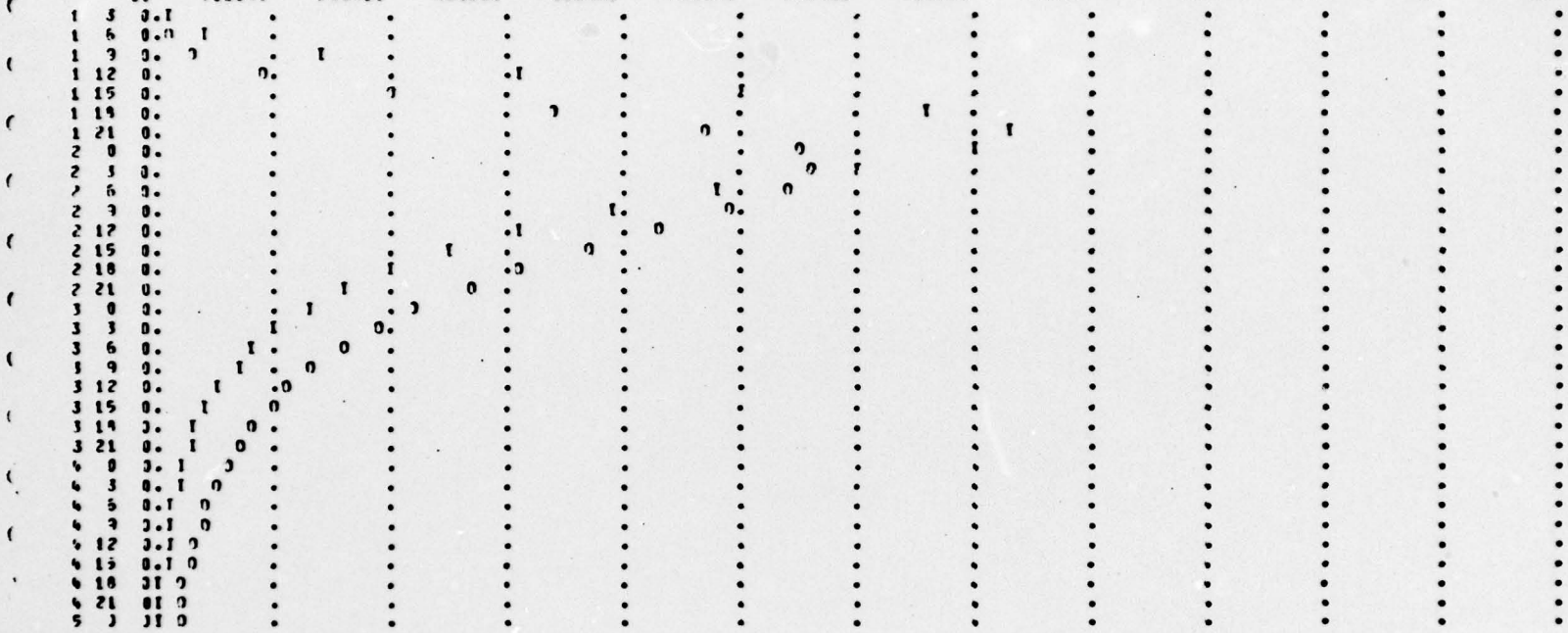
3	9	0	14751.	6657.	13346.
3	12	0	11041.	5342.	11126.
3	15	0	11544.	4286.	9523.
3	18	0	10213.	3433.	8037.
3	21	0	9044.	2760.	6946.
4	0	0	8031.	2215.	5960.
4	3	0	7159.	1777.	4926.
4	6	0	6415.	1426.	4029.
4	9	0	5754.	1144.	3588.
4	12	0	5140.	919.	3205.
4	15	0	4564.	706.	2946.
4	18	0	3986.	331.	2685.
4	21	0	3421.	34.	2134.
5	0	0	2930.	0.	1927.

SUM 603121.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	56659.	55619.	46787.	24617.	603121.
INCHES		1.40	4.71	7.17	7.59
AC-FT		27594.	92948.	145369.	149661.

STATION 2

INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)



RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	73904.	72815.	55404.	25227.	170.00
ROUTED TO	2	56451.	55613.	46282.	24417.	170.00

STABILITY ANALYSES

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. _____ OF _____

NAME OF CLIENT NYSDEC

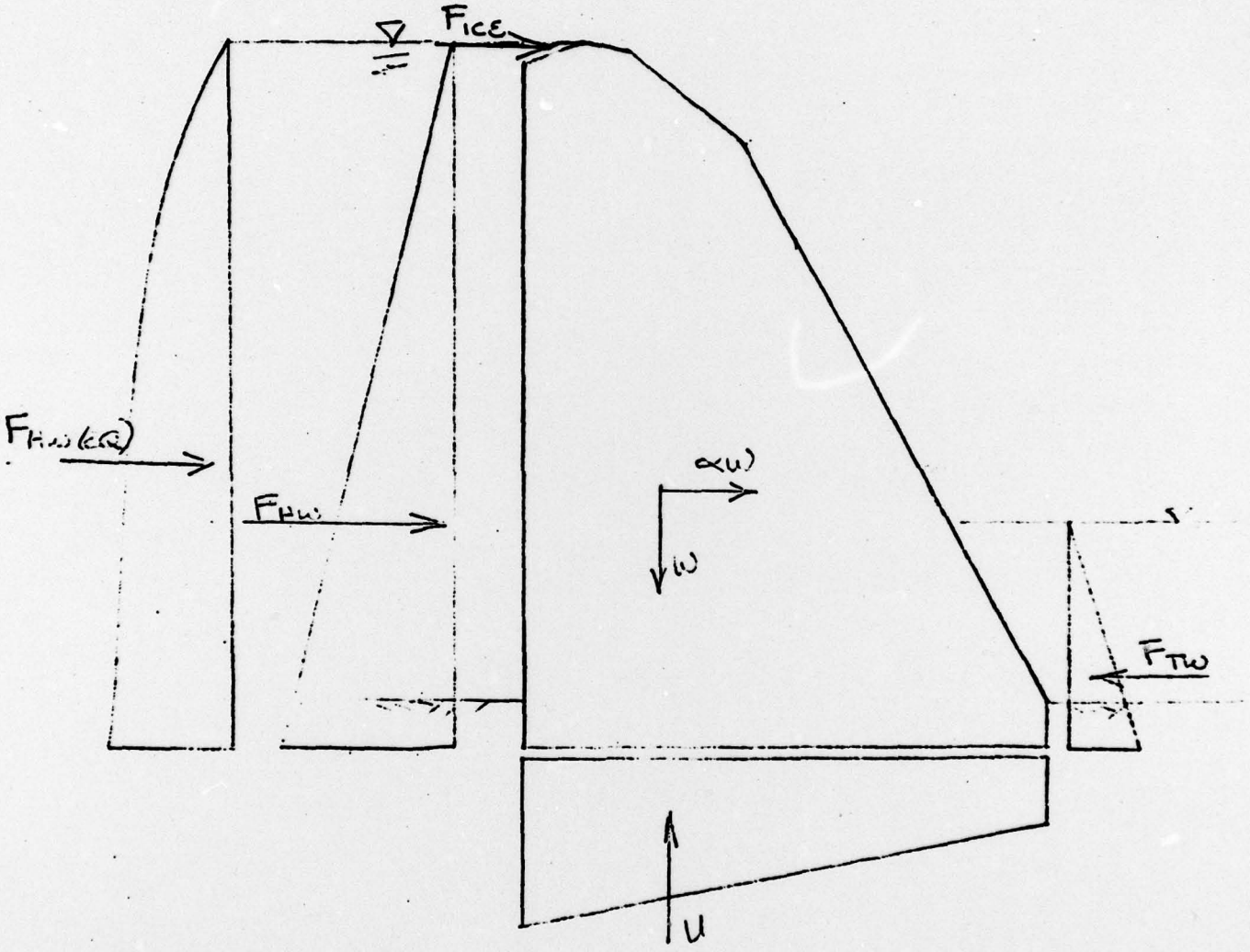
DATE 7/1/80

PROJECT Hinckley Reservoir

COMP. BY DBC

CHECKED BY _____

GENERALIZED LOADING FOR SPILLWAY



W - Weight of Section
F_{HW} - Headwater Force
F_{TW} - Tailwater Force
$F_{HW(EQ)}$ - Earthquake inertia of Headwater
αW - Earthquake inertia of Gravity Section
U - Uplift
F_{ICE} - Ice Load

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. _____ OF _____

NAME OF CLIENT N.Y.DEC.

DATE 6/27/78

COMP. BY DEC

PROJECT HINCKLEY RESERVOIR DAM

CHECKED BY _____

Effective strength of anchor bolts through the keyway of the spillway section.

Embedment length = 8' Bar size = 2.5"

Pullout shear area = $\pi \times d \times L$

$= \pi \times 2.5" \times 8' \times 12"/ft.$

$= 754 \square"$

Assume 100 psi shear needed to break the bond.

Shear per bar = $100 psi \times 754 \square" = 75{,}400\#$

Bars are spaced at 8' c-c, so the available shear/foot section = $9{,}425\# \approx 9.4^k$

Stabilizing moment/foot section = $9.4^k \times 57'$

$= 536'^k$

Per foot force reqd at 57' for PMF to be in the central third of the base.

$$\frac{3413 + 57 \times F_{AE}}{282.5 + F_{AE}} = 62/3 \qquad F_{AE} = 66.75^{k/ft}$$

or 534^k per bar.

STABILITY ANALYSIS OF MASONRY SPILLWAY AT HINCKLEY RESERVOIR
NORMAL POOL

BASE ELEVATION= 1142.00FT. TOP ELEVATION= 1225.00FT. BASE WIDTH= 62.00FT. DENSITY= 155.00PCF
HEADWATER ELEVATION= 1225.00FT. TAILWATER ELEVATION= 1155.00FT. EARTHQUAKE ACCELERATION***.000G (HORIZ)..000G (VERT)
SILT ELEVATION= 0.00FT. SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 100.00PSI SHEAR WIDTH= 62.00FT. FRICTION FACTOR= .60

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	531.24	38.89	20657.44	
HEADWATER	214.94	27.64		5940.64
TAILWATER	5.27	4.33	22.81	
UPLIFT	185.78	38.53		7155.99
			**********	**********
			20680.26	13096.63

NET HORIZONTAL FORCE= 209.66 KIPS
NET VERTICAL FORCE= 345.54 KIPS
NET MOMENT= 7583.64KIP-FEET
X-BAR OF FOUNDATION REACTION= 21.95 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 9.05 FEET
FOUNDATION REACTION PRESSURES********TOE= 72.61 PSI********HEEL= 4.80 PSI********
OVERTURNING FACTOR OF SAFETY= 1.58
SLIDING FACTOR OF SAFETY= .99
DEVELOPED FRICTION FACTOR (NO SHEAR)= .61
SLIDING WITH SHEAR FACTOR OF SAFETY= 5.25(SHEAR ACROSS FULL BASE WIDTH)

STABILITY ANALYSIS OF MASONRY SPILLWAY AT HINCKLEY RESERVOIR
NORMAL POOL AND ICE LOAD

BASE ELEVATION= 1142.00FT. TOP ELEVATION= 1225.00FT. BASE WIDTH= 62.00FT. DENSITY= 155.00PCF
HEADWATER ELEVATION= 1225.00FT. TAILWATER ELEVATION= 1155.00FT. EARTHQUAKE ACCELERATION***.000G (HORIZ)..000G (VERT)
SILT ELEVATION= 0.00FT. SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 100.00PSI SHEAR WIDTH= 62.00FT. FRICTION FACTOR= .60

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	531.24	38.89	20657.44	
HEADWATER	214.94	27.64		5940.64
TAILWATER	5.27	4.33	22.83	
UPLIFT	185.78	38.53		7155.99
ICE LOAD	5.00	83.00		415.00
			20680.26	13511.63

NET HORIZONTAL FORCE= 214.66 KIPS
NET VERTICAL FORCE= 345.54 KIPS
NET MOMENT= 7168.64KIP-FEET
X-BAR OF FOUNDATION REACTION= 20.75 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 10.25 FEET
FOUNDATION REACTION PRESSURES********TOE= 77.11 PSI********HEEL= .38 PSI********
OVERTURNING FACTOR OF SAFETY= 1.53
SLIDING FACTOR OF SAFETY= .97
DEVELOPED FRICTION FACTOR (NO SHEAR)= .62
SLIDING WITH SHEAR FACTOR OF SAFETY= 5.12(SHEAR ACROSS FULL BASE WIDTH)

STABILITY ANALYSIS OF MASONRY SPILLWAY AT HINCKLEY RESERVOIR
NORMAL POOL AND EARTHQUAKE

BASE ELEVATION= 1142.00FT. TOP ELEVATION= 1225.00FT. BASE WIDTH= 62.00FT. DENSITY= 155.00PCF
HEADWATER ELEVATION= 1225.00FT. TAILWATER ELEVATION= 1155.00FT. EARTHQUAKE ACCELERATION***.050G (HORIZ)..000G (VERT)
SILT ELEVATION= 0.00FT, SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 100.00PSI SHEAR WIDTH= 62.00FT. FRICTION FACTOR= .60

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	531.24	38.89	20657.44	
HEADWATER	214.94	27.64		5940.64
TAILWATER	5.27	4.33	22.83	
UPLIFT	185.78	38.53		7155.99
EARTHQUAKE INDUCED LOADINGS				
INERTIA-WATER	11.71	33.20		388.82
HORIZONTAL INERTIA-DAM	26.56	33.34		885.47
			20680.26	14370.91

NET HORIZONTAL FORCE= 247.94 KIPS
NET VERTICAL FORCE= 345.54 KIPS
NET MOMENT= 6309.35KIP-FEET
X-BAR OF FOUNDATION REACTION= 18.26 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 12.74 FEET
**********FOUNDATION REACTION NOT IN CENTRAL THIRD OF BASE*****TENSION AT HEEL OF DAM*****
FOUNDATION REACTION PRESSURES********TOE= 86.42 PSI********HEEL= -9.82 PSI********
OVERTURNING FACTOR OF SAFETY= 1.44
SLIDING FACTOR OF SAFETY= .84
DEVELOPED FRICTION FACTOR (NO SHEAR)= .72
SLIDING WITH SHEAR FACTOR OF SAFETY= 4.44(SHEAR ACROSS FULL BASE WIDTH)

****

STABILITY ANALYSIS OF MASONRY SPILLWAY AT HINCKLEY RESERVOIR
PROBABLE MAXIMUM FLOOD

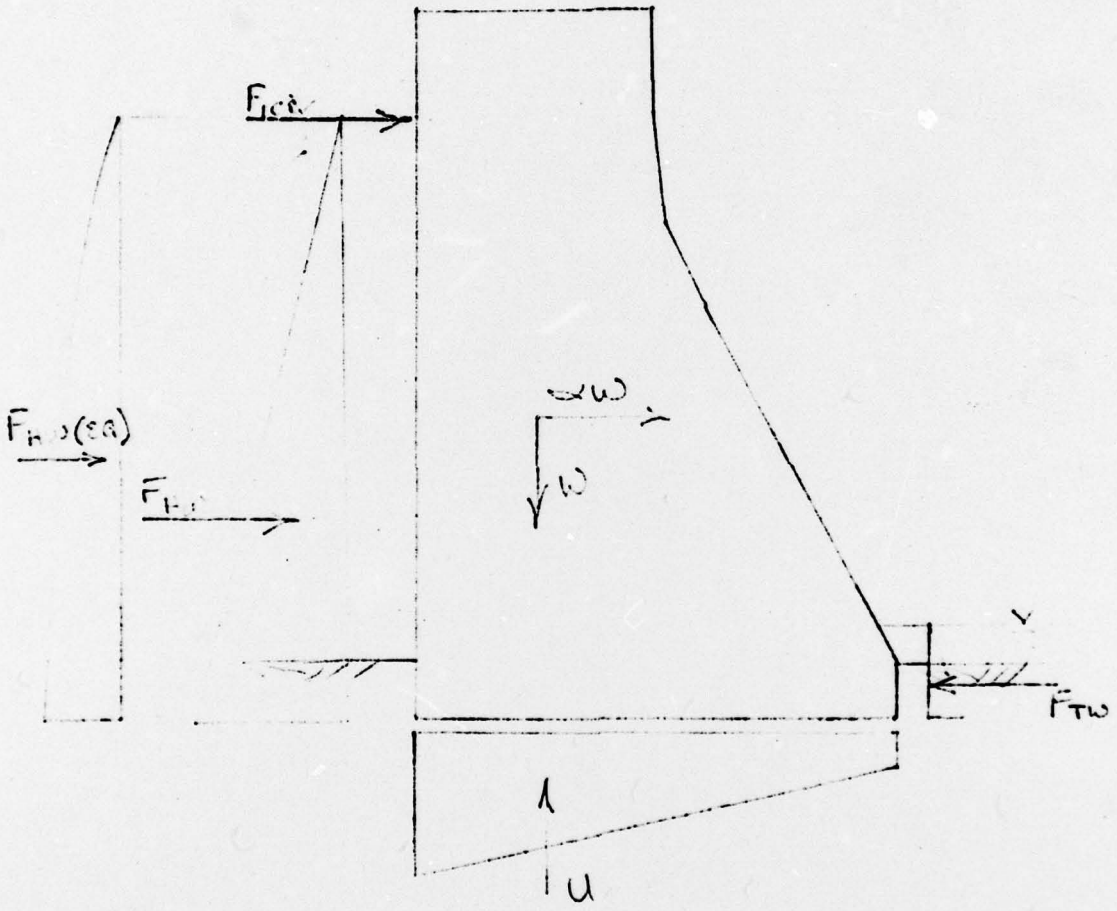
****

BASE ELEVATION= 1142.00FT. TOP ELEVATION= 1225.00FT. BASE WIDTH= 62.00FT. DENSITY= 155.00PCF
HEADWATER ELEVATION= 1237.60FT. TAILWATER ELEVATION= 1175.00FT. EARTHQUAKE ACCELERATION=.000G (HORIZ),.000G (VERT)
SILT ELEVATION= 0.00FT. SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 100.00PSI SHEAR WIDTH= 62.00FT. FRICTION FACTOR= .60

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	531.24	38.89	20657.44	
HEADWATER	280.19	30.89		8654.79
TAILWATER	33.98	10.99	373.37	
UPLIFT	248.76	36.03		8962.98
			**********	**********
			21030.81	17617.77

NET HORIZONTAL FORCE= 246.22 KIPS
NET VERTICAL FORCE= 282.48 KIPS
NET MOMENT= 3413.04KIP-FEET
X-BAR OF FOUNDATION REACTION= 12.08 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 18.92 FEET
**********FOUNDATION REACTION NOT IN CENTRAL THIRD OF BASE*****TENSION AT HEEL OF DAM*****
FOUNDATION REACTION PRESSURES********TOE= 69.56 PSI********HEEL= -28.28 PSI********
OVERTURNING FACTOR OF SAFETY= 1.19
SLIDING FACTOR OF SAFETY= .69
DEVELOPED FRICTION FACTOR (NO SHEAR)= .87
SLIDING WITH SHEAR FACTOR OF SAFETY= 4.31(SHEAR ACROSS FULL BASE WIDTH)

GENERALIZED LOADING FOR NONOVERFLOW SECTIONS



W - weight of section
F_{HW} - headwater force
F_{TW} - tailwater force
$F_{HW(EQ)}$ - earthquake increment of headwater
αW - earthquake inertia of gravity section
U - uplift
F_{ICE} - ice load

STABILITY ANALYSIS OF MASONRY GRAVITY SECTION AT HINCKLEY RESERVOIR
NORMAL POOL

BASE ELEVATION= 1162.00FT. TOP ELEVATION= 1240.00FT. BASE WIDTH= 51.50FT. DENSITY= 155.00PCF
HEADWATER ELEVATION= 1225.00FT. TAILWATER ELEVATION= 0.00FT. EARTHQUAKE ACCELERATION=***.0006 (HORIZ)..0006 (VERT)
SILT ELEVATION= 0.00FT. SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 100.00PSI SHEAR WIDTH= 51.50FT, FRICTION FACTOR= .60

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	432.02	32.65	14050.31	
HEADWATER	123.83	20.98		2597.89
UPLIFT	101.23	34.33		3475.51
			14050.31	6073.40

NET HORIZONTAL FORCE= 123.83 KIPS
NET VERTICAL FORCE= 331.70 KIPS
NET MOMENT= 7976.91KIP-FEET
X-BAR OF FOUNDATION REACTION= 24.05 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 1.70 FEET
FOUNDATION REACTION PRESSURES*******TOE= 51.59 PSI********HEEL= 35.86 PSI********
OVERTURNING FACTOR OF SAFETY= 2.31
SLIDING FACTOR OF SAFETY= 1.61
DEVELOPED FRICTION FACTOR (NO SHEAR)= .37
SLIDING WITH SHEAR FACTOR OF SAFETY= 7.60(SHEAR ACROSS FULL BASE WIDTH)

STABILITY ANALYSIS OF MASONRY GRAVITY SECTION AT HINCKLEY RESERVOIR
NORMAL POOL AND ICE LOAD

BASE ELEVATION= 1162.00FT. TOP ELEVATION= 1240.00FT. BASE WIDTH= 51.50FT. DENSITY= 155.00PCF
HEADWATER ELEVATION= 1225.00FT. TAILWATER ELEVATION= 0.00FT. EARTHQUAKE ACCELERATION=**.000G (HORIZ)..000G (VERT)
SILT ELEVATION= 0.00FT. SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 100.00PSI SHEAR WIDTH= 51.50FT. FRICTION FACTOR= .60

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	432.92	32.45	14050.31	
HEADWATER	123.83	20.98		2597.89
UPLIFT	101.23	34.33		3475.51
ICE LOAD	5.50	63.00		315.00
			14050.31	6388.40

NET HORIZONTAL FORCE= 158.83 KIPS
NET VERTICAL FORCE= 331.70 KIPS
NET MOMENT= 7661.91KIP-FEET
X-BAR OF FOUNDATION REACTION= 23.10 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 2.65 FEET
FOUNDATION REACTION PRESSURES*******TOE= 58.54 PSI*******HEEL= 30.91 PSI*******
OVERTURNING FACTOR OF SAFETY= 2.20
SLIDING FACTOR OF SAFETY= 1.54
DEVELOPED FRICTION FACTOR (NO SHEAR)= .39
SLIDING WITH SHEAR FACTOR OF SAFETY= 7.30(SHEAR ACROSS FULL BASE WIDTH)

STABILITY ANALYSIS OF MASONRY GRAVITY SECTION AT HINCKLEY RESERVOIR
NORMAL POOL AND EARTHQUAKE

BASE ELEVATION= 1162.00FT. TOP ELEVATION= 1240.00FT. BASE WIDTH= 51.50FT. DENSITY= 155.00PCF
HEADWATER ELEVATION= 1225.00FT. TAILWATER ELEVATION= 0.00FT. EARTHQUAKE ACCELERATION=.050G (HORIZ)..000G (VERT)
SILT ELEVATION= 0.00FT. SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 100.00PSI SHEAR WIDTH= 51.50FT. FRICTION FACTOR= .60

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	432.32	32.45	14050.31	
HEADWATER	123.83	20.98		2597.89
UPLIFT	101.23	34.33		3475.51
EARTHQUAKE INDUCED LOADINGS				
INERTIA-WATER	7.61	25.20		189.19
HORIZONTAL INERTIA-DAM	21.65	33.51		725.46
			14050.31	6988.05

NET HORIZONTAL FORCE= 152.99 KIPS
NET VERTICAL FORCE= 331.70 KIPS
NET MOMENT= 7062.26KIP-FEET
X-BAR OF FOUNDATION REACTION= 21.29 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 4.46 FEET
FOUNDATION REACTION PRESSURES TOE= 67.96 PSI HEEL= 21.49 PSI
OVERTURNING FACTOR OF SAFETY= 2.01
SLIDING FACTOR OF SAFETY= 1.30
DEVELOPED FRICTION FACTOR (NO SHEAR)= .46
SLIDING WITH SHEAR FACTOR OF SAFETY= 6.15(SHEAR ACROSS FULL BASE WIDTH)

STABILITY ANALYSIS OF MASONRY GRAVITY SECTION AT PINCKLEY RESERVOIR
PROBABLE MAXIMUM FLOOD

BASE ELEVATION= 1162.00FT. TOP ELEVATION= 1240.00FT. BASE WIDTH= 51.50FT. DENSITY= 155.00PCF
HEADWATER ELEVATION= 1237.60FT. TAILWATER ELEVATION= 1175.00FT. EARTHQUAKE ACCELERATION=**.000G (HORIZ)+.000G (VERT)
SILT ELEVATION= 0.00FT. SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 100.00PSI SHEAR WIDTH= 51.50FT. FRICTION FACTOR= .60

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	432.02	32.45	14050.31	
HEADWATER	178.32	25.17		4489.15
TAILWATER	5.27	4.33	22.83	
UPLIFT	142.36	31.81		4529.19
			14073.13	9018.35

NET HORIZONTAL FORCE= 173.05 KIPS

NET VERTICAL FORCE= 240.56 KIPS
NET MOMENT= 5054.79KIP-FEET
X-BAR OF FOUNDATION REACTION= 17.40 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 8.35 FEET
FOUNDATION REACTION PRESSURES********TOE= 77.31 PSI********HEEL= 1.05 PSI********
OVERTURNING FACTOR OF SAFETY= 1.56
SLIDING FACTOR OF SAFETY= 1.01
DEVELOPED FRICTION FACTOR (NO SHEAR)= .60
SLIDING WITH SHEAR FACTOR OF SAFETY= 5.29(SHEAR ACROSS FULL BASE WIDTH)